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A SERIES OF FAILURE TESTS OF FULL-SIZE COMPRESSION MEMBERS, MADE FOR THE PENNSYLVANIA LINES WEST OF PITTSBURG.

By C. P. BUCHANAN.*

The making of tests of full-size compression members of steel railway bridges was undertaken by the engineering department of the Pittsburg, Cincinnati, Chicago & St. Louis Railway, some twenty years ago and has been continued at intervals since that time. The results obtained have been of so much value to those interested in the bridge work on the Pennsylvania Lines that it was thought they might interest other bridge engineers if published. The lessons learned have been many and of a somewhat progressive nature. For the purpose of this article, however, it will suffice to give the facts observed and permit any one that cares to do so to make his own analysis.

It may be well, however, to call attention to the fact that the earlier tests demonstrated that the usual practice of attaching pin plates to webs with enough rivets to transmit the proportion of load they were to carry would not insure a detail that was "as strong as the member" unless the rivets were so placed as to transmit from these plates to the flanges direct that portion of the load they, the flanges, were designed to carry; and that a compression member having one or two sides latticed is likely to fail under a smaller load than indicated by any accepted formula.

All of the tests were made at the works of the Keystone Bridge Co., Pittsburg, Pa., in a hydraulic machine that was manufactured by Wm. Sellers & Co., under an order from Mr. Linville, while he was in charge of the bridge department of the Pennsylvania Railroad. This was originally an accumulator-operated machine, of about 500,000 lbs. capacity, but had had the frame strengthened and a gang of plunger pumps attached that were used to supplement the accumulator when loads beyond its limit were to be applied.

The members were all placed in the machine in a horizontal position, carefully leveled, and counterweighted at the center by a weight sufficient to balance one-half the weight of the test column (except in test III).

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An extensometer designed by Mr. G. C. Henning, reading by vernier to 0.0001-in., was used for measuring the change of length of the member under loads; the attachment was in all cases to the *centers of the end pins* of the column. The deflections were noted on verniers connected to the center of column, showing the departure from a longitudinal line through centers of end bearings to the nearest 0.005 in.

The elastic value in compression was taken as that load under which the rate of compression per unit increased and the yield point noted was where there was a distinct permanent set or apparent tendency to fail. The deflection movements, lateral and vertical, were so irregular and eccentric that they were not thought to have any particular significance in determining the elastic value of the column.

To insure the best bearings and alinement in the machine, a load of 25,000 lbs. was repeatedly applied to each test column before any measurements were recorded, and in determining the compression in inches per inch, for computing the modulus of elasticity in compression, care was exercised to avoid the use of any readings below 25,000 lbs. or beyond the load where the rate of compression per unit increased.

A TEST COLUMN IN POSITION FOR TEST IN THE KEYSTONE TESTING MACHINE.

The tests are recorded individually below, in the order in which they were made. A tabular summary of the principal data is given as Table I. The construction of the various members tested is shown in the drawings given with the individual tests, and their appearance after failure is shown by photographs.

To avoid forming erroneous conclusions from the appearance of the tested members in the photographs, it should be remarked that after failure of a member, and after the removal of extensometer and deflection verniers, load was applied until the column was distorted as much as the frame of the testing-machine would permit, so that the character of the failure would show as distinctly as possible in the photograph.

The first four tests reported were made before securing an extensometer that would show with sufficient accuracy the measurements of compression, and consequently the record gives only the maximum load and elastic limit, with description of the kind of failure. For all succeeding tests the total compression and set, and the vertical and horizontal deflection, both total and permanent, are recorded.

I. WROUGHT-IRON INTERMEDIATE POST, Bridge 44, C. & M. V. RY. Tested Dec. 26, 1888.

Z-bar column, as per Fig. 1, four 5-in. Z-bars and one 7 × ⅜-in. plate; pin ends.

$A = 17.78$ sq. ins.; $r = 3.05$ ins. (across pin);

$l/r = 98$.

First well-defined indication of permanent set was at 18,000 lbs. per sq. in. Maximum load 24,800 lbs. per sq. in.

Loads.	Deflection in Inches. Vertical. Load.	Vertical. Set.	Horizontal. Load.	Horizontal. Set.
440,000	0.31	0.16	1.03	0.26

Failed suddenly at 440,700 lbs.

The deflection was to the left (looking from base to top of post) and upward. After failure the deflection was 2.23 ins. to left and 0.21-in. up.

Fig. 1a shows the column after failure (left-hand column).

II. WROUGHT-IRON INTERMEDIATE POST, BRIDGE 50, C., ST. L. & P. R. R. Tested Dec. 26, 1888.

Z-bar column, as per Fig. 2; four 4-in. Z-bars and one 6 × ⅜-in. plate.

$A = 17.56$ sq ins.; $r = 2.49$ ins. (across pin);

$l/r = 120$.

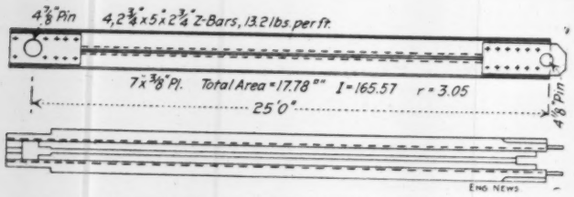


FIG. 1. INTERMEDIATE POST, BRIDGE 44, C. & M. V. RY.

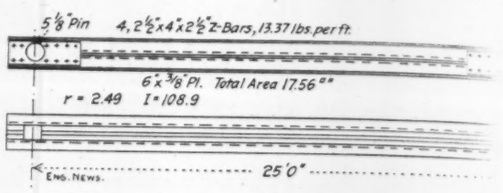


FIG. 2. INTERMEDIATE POST, BRIDGE 50, C., ST. L. & P.

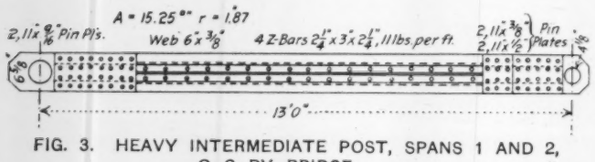


FIG. 3. HEAVY INTERMEDIATE POST, SPANS 1 AND 2, O. C. RY. BRIDGE.

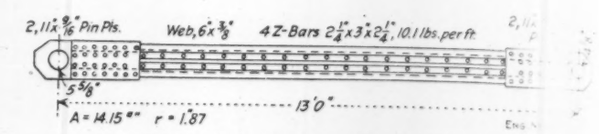


FIG. 4. LIGHT INTERMEDIATE POST, SPANS 1 AND 2, O. C. RY. BRIDGE.

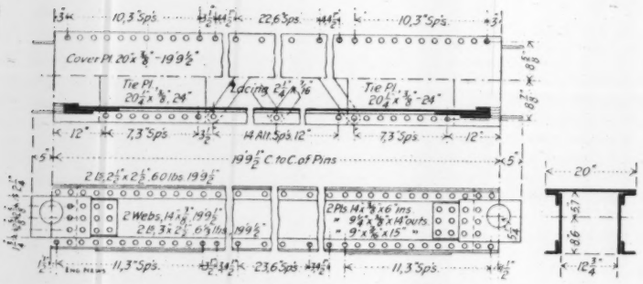


FIG. 5. TOP CHORD, BRIDGE 75, CINCINNATI DIVISION, P., C., C. & ST. L. RY.

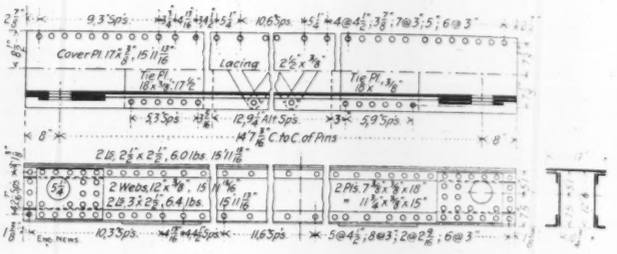


FIG. 6. TOP CHORD, BRIDGE 25, C., R. & C. R. R.

First well-defined indication of permanent set was at 20,500 lbs. per sq. in. Maximum load 26,350 lbs. per sq. in.

Loads.	Deflection in Inches. Vertical. Load.	Vertical. Set.	Horizontal. Load.	Horizontal. Set.
460,000	0.38	0.23	....	0.31
462,800	0.58	....	....	....

Failed suddenly, breaking vernier connection.

The deflection was to the left and downward. The column failed by deflecting to left at right angles to pin, in direction of least radius of gyration.

Fig. 1a shows the column after failure (right-hand column).

Fig. 1a. View After Failure; Wrought-Iron Z-Bar Posts, Bridge 44, C. & M. V. Ry., and Bridge 50, C., St. L. & P. R. R.
Maximum loads, 24,800 and 26,350 lbs. per sq. in.

III. STEEL INTERMEDIATE POST, SPANS 1 AND 2, O. C. RY. BRIDGE. Tested June 24, 1890.

Z-bar column, as per Fig. 3; four 3-in. Z-bars and one 6 × ⅜-in. plate. Material: Bessemer steel.

$A = 15.25$; $r = 1.87$ ins. (about pin); $l/r = 83$.

Column tested with pins vertical, and counterweight attached at center equal to whole weight of column.

Apparent permanent set at 23,600 lbs. per sq. in. Maximum load 34,270 lbs. per sq. in.

Loads.	Deflection in Inches. Vertical. Load.	Vertical. Set.	Horizontal. Load.	Horizontal. Set.
500,000	—0.22	—0.06	—0.05	—0.01
522,600	—1.44	—1.05	+1.01	+1.01

Deflection downward. After failure, there was a uniform curve along half the length of the post midway between pin-plates, the pin-holes were slightly elongated and rivets in pin-plates loosened on convex side. Pins were slightly marked by bearing-plates.

IV. STEEL INTERMEDIATE POST, SPANS 1 AND 2, O. C. BRIDGE. Tested June 24, 1890.

Z-bar column as per Fig. 4; four 3-in. Z-bars and one 6 × ⅜-in. plate. Material, Bessemer steel.

$A = 14.15$; $r = 1.87$ ins. (about pin); $l/r = 83$.

Column tested in same manner as III., except that the counterweight was made to balance only half the weight of the column.

Indications of permanent set at 24,700 lbs. per sq. in. Maximum load 32,900 lbs. per sq. in.

Loads.	Deflection in Inches. Vertical. Load.	Vertical. Set.	Horizontal. Load.	Horizontal. Set.
450,000	0.09	0.07	0.09	0.06
465,400	0.93	0.55	0.35	0.32

Deflection upward and to right, column bending to a circular curve for about half the length of the column along center. Pin-holes slightly elongated and rivets in pin-plates loosened on convex side. Pins slightly indented by bearing-plates.

V. WROUGHT-IRON TOP CHORD FROM BRIDGE 75, CINCINNATI DIV., P., C., C. & ST. L. RY. Tested Aug. 7, 1891.

Chord as shown by Fib. 5.* Material, wrought iron, elastic limit 30,630-37,675, ult. strength 51,470-55,290 lbs. per sq. in.

$A = 25.4$ sq. ins.; $r = 5.55$ ins. (about pin); $l/r = 43$.

Loads.	Compression in Inches. Load.	Compression in Inches. Set.	Deflection in Inches. Vertical. Load.	Vertical. Set.	Horizontal. Load.	Horizontal. Set.
26,000	.0345	.0025	+.01	...	+.01	...
50,700	.0415	.0020	+.015	...	—.01	...
75,400	.0485	.0028	+.02	...	0.	...
100,100	.0575	.0020	+.025	...	+.01	...
124,800	.0685	.0030	+.03	...	+.01	0.
149,500	.0790	.0034	+.04	...	+.01	...
175,500	.0875	.0038	+.05	...	+.01	...
200,200	.0990	.0034	+.055	...	+.01	...
224,900	.1095	.0040	+.055	...	+.01	...
249,600	.1220	.0048	+.070	...	+.01	...
275,600	.1330	.0050	+.075	...	+.01	...
300,300	.1450	.0065	+.08	...	+.01	...
325,000	.1560	.0115	+.08	...	+.01	...
349,700	.1675	.0145	+.085	...	+.02	...
374,400	.1810	.0188	+.090	...	+.02	...
400,400	.1983	.0279	+.095	...	+.02	...
425,100	.2180	.0346	+.1	...	+.03	+.01
449,800	.2365	.0450	+.105	+.035	+.03	+.02
460,200	.2450	.0570	+.11	...	+.04	...
470,600	.2610	.0637	+.11	+.030	+.04	...
479,700	.2700	.0675	+.105	...	+.05	+.02
490,100	.2815	.0759	+.105	...	+.05	...
500,500	.3030	.0880	+.1	...	+.06	+.03
509,600	.3200	.1195	+.105	+.03	+.06	...
520,000	.3432	.1258	+.1	+.025	+.06	...
530,400	.3588	.1306	+.095	+.02	+.06	...
539,500	.3800	.1509	+.09	+.01	+.07	...
549,900	.4025	.1760	+.08	+.005	+.07	...
560,300	.4315	.2035	+.07	—.015	+.07	+.03
569,400	.4635	.2905	+.055	—.03	+.07	...
579,800	.5038	.3311	+.03	—.06	+.07	...
590,200	.5525	.4407	—.015	—.120	+.07	+.03
591,500	....	....	—.09	...	+.05	...

Failed by buckling of web-plates at inner end of pin-plates, the four points failing at the same time, and web and inner pin-plate tearing open at one point.

Elastic limit of column 13,800 lbs. per sq. in. Maximum strength 23,200 lbs. per sq. in. The compression and the lateral and vertical deflections, plotted on load, are shown in Fig. 5a. The appearance of the member after failure is shown by Fig. 5b.

*The directions of deflection, "Vertical" and "Horizontal," in all the following tables, are to be interpreted thus: In tests of chords, "Vertical" means transverse to pin, "Horizontal" means parallel to pin. In tests of posts IX. and XVII., "Vertical" means parallel to pin, "Horizontal" means transverse to pin.

All tests were made with pins vertical in the testing-machine, thus making the chord-members lie sidewise. The deflections were referred to the position these members would have in the bridge.

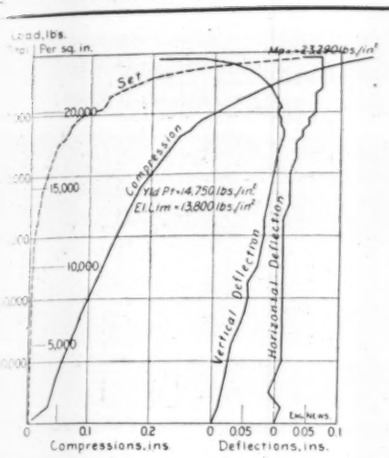


Fig. 5a. Compression, Set and Transverse Deflections; Wrought-Iron Top Chord, Bridge 75, Cincinnati Division, P., C., C. & St. L. Ry.

VI. WROUGHT-IRON TOP CHORD, BRIDGE 25, C., R. & C. R. R. Tested Aug. 8, 1891.

Material, wrought iron, el. lim. 28,360-33,050, ult. strength 47,770-52,430 lbs. per sq. in. Chord built as per Fig. 6.

$A = 22.8$ sq. ins.; $r = 4.88$ ins. (about pin); $l/r = 36$.

Loads.	Compression in Inches.		Deflection in Inches. Vertical.		Horizontal.	
	Load.	Set.	Load.	Set.	Load.	Set.
26,000	.0003	.0	+.01	0	0	...
50,700	.0058	.0021	+.015	+.01	0	...
75,400	.0149	.0029	+.02	+.01	0	...
100,100	.0248	.0032	+.025	+.01	0	...
124,800	.0318	.0035	+.03	+.01	0	...
149,500	.0423	.0038	+.03	+.01	0	...
175,500	.0528	.0048	+.03	+.01	0	...
200,200	.0640	.0063	+.03	+.01	0	...
224,900	.0713	.0082	+.035	+.005	0	...
249,600	.0843	.0094	+.04	+.005	0	...
275,600	.0949	.0128	+.04	+0	0	...
300,300	.1053	.0141	+.045	0	0	...
325,000	.1158	.0166	+.045	0	0	...
349,700	.1293	.0200	+.05	0	+.01	—.005
374,400	.1438	.0262	+.055	0	+.01	—.005
400,400	.1616	.0343	+.055	0	+.01	—.005
409,500	.1693	.0382	+.06	+.005	+.01	—.005
419,900	.1803	.0420	+.06	+.005	+.01	—.005
430,300	.1918	.0456	+.06	0	+.01	—.005
439,400	.2013	.0508	+.065	0	—.005	—.005
449,800	.2078	.0547	+.065	0	—.005	—.005
460,200	.2163	.0612	+.07	+.005	+.01	—.005
470,600	.2203	.0681	+.07	0	+.01	—.005
479,700	.2293	.0754	+.07	0	+.01	—.005
490,100	.2410	.0825	+.07	0	0	—.005
500,500	.2603	.0953	+.07	0	0	—.005
509,600	.2691	.1017	+.07	0	0	—.005
520,000	.2835	.1119	+.07	0	0	—.005
530,400	.2938	.1198	+.07	—.005	—.005	—.005
539,500	.3111	.1324	+.07	—.005	0	—.005
549,900	.3288	.1466	+.07	—.01	0	—.005
574,600	.3828	.1910	+.07	—.015	—.005	—.005
600,600	.4403	.2378	+.07	—.015	—.005	—.005
625,300	.5338	.3203	+.07	—.02	—.005	—.005
643,500	.7428	.5878	+.065	—.12	0	0

Failed by buckling of web-plates at inner end of pin-plates at plunger end, web and pin-plates tearing through pin-hole.

Elastic limit 13,200 lbs. per sq. in. Maximum strength 28,220 lbs. per sq. in. The compression and the lateral and vertical deflections, plotted on load, as shown in Fig. 6a. The appearance of the member after failure is shown by Fig. 6b.

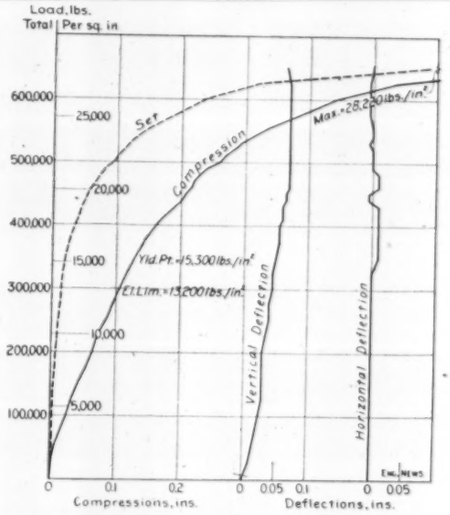


Fig. 6a. Compression, Set and Transverse Deflections; Wrought-Iron Top Chord, Bridge 25, C., R. & C. R. R.

VII. WROUGHT-IRON TOP CHORD, BRIDGE 1, T., W. V. & O. R. R. Tested May 19, 1892.

Material, wrought iron, el. lim. 29,360-30,790, ult. strength 49,530-53,340 lbs. per sq. in. Chord built as shown by Fig. 7.

$A = 27.95$ sq. ins.; $r = 5.69$ ins. (about pin); $l/r = 39$.

Loads.	Compression in Inches.		Deflection in Inches. Vertical.		Horizontal.	
	Load.	Set.	Load.	Set.	Load.	Set.
49,400	.0610	.0038	+.005	0.	.0	.0
100,100	.0835	.0060	+.005	0.	—.010	.0
149,500	.1022	.0135	+.010	0	—.010	.0
200,200	.1235	.0175	+.025	+.005	—.020	.0
224,900	.1323	.0203	+.030	+.005	—.020	.0
249,600	.1422	.0180	+.040	+.005	—.020	.0
275,600	.1514	.0230	+.040	+.005	—.020	—.005
300,300	.1618	.0296	+.045	+.005	—.020	—.005
325,000	.1703	.0335	+.050	+.005	—.040	—.005
349,700	.1798	.0360	+.055	+.005	—.035	—.005
374,400	.1930	.0398	+.055	+.005	—.040	—.005
400,400	.2010	.0306	+.060	+.005	—.040	—.005
425,100	.2105	.0360	+.065	+.005	—.040	—.005
449,800	.2235	.0440	+.070	+.005	—.045	—.005
474,500	.2348	.0458	+.080	+.005	—.040	.0
500,500	.2470	.0535	+.085	+.005	—.040	.0
525,200	.2590	.0510	+.090	+.005	—.040	.0
549,900	.2730	.0535	+.095	+.005	—.045	.0
574,600	.2930	.0803	+.095	+.005	—.050	.0
600,600	.3125	.0681	+.100	+.005	—.050	.0
650,000	.3675	.0935	+.095	—.010	—.055	.0
699,400	.4410	.1271	+.085	—.035	—.070	.0
750,100	.5328	.2360	+.070	—.050	—.090	.0
799,500	.6910	.3857	+.115	—.015	—.120	—.010
816,400	.8060	1.1600	+.145	...	—.210	...

Failed by webs shearing rivets through ends of flange-angles and buckling at ends of pin-plates, at plunger end. At tail-block end some rivets through webs and angles were loosened.

Elastic limit 15,200 lbs. Maximum strength 29,210 lbs. The compression and the lateral and vertical deflections, plotted on load, are shown in Fig. 7a. The appearance of the member after failure is shown by Fig. 7b.

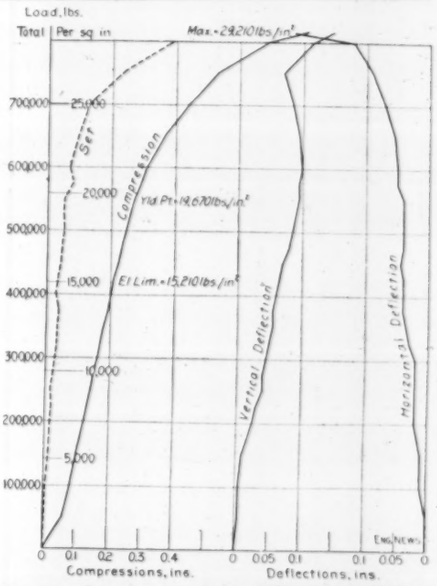


Fig. 7a. Compression, Set and Transverse Deflections; Wrought-Iron Top Chord, Bridge 1, T., W. V. & O. Ry.

VIII. WROUGHT-IRON TOP CHORD BRIDGE 3, T., W. V. & O. R. R. Tested May 21, 1892.

Material wrought iron, el. lim. 29,360-30,790, ult. strength 49,530-53,340 lbs. per sq. in. Chord built as per Fig. 8.

$A = 27.95$ sq. ins.; $r = 5.69$ ins. (about pin); $l/r = 39$.

Loads.	Compression in Inches.		Deflection in Inches. Vertical.		Horizontal.	
	Load.	Set.	Load.	Set.	Load.	Set.
49,400	.0097	—.0065	+.015	0.	—.020	0.
100,100	.1283	—.0033	+.035	0.	—.030	0.
149,500	.1420	+.0020	+.045	+.005	—.050	0.
200,200	.1570	.0024	+.065	+.010	—.050	0.
224,900	.1657	.0030	+.080	+.015	—.060	0.
249,600	.1732	.0020	+.090	+.015	—.060	0.
275,600	.1835	.0060	+.100	+.025	—.060	0.
300,300	.1920	.0088	+.110	+.025	—.060	0.
325,000	.2005	.0100	+.115	+.030	—.065	0.
349,700	.2090	.0137	+.120	+.030	—.070	0.
374,400	.2180	.0133	+.125	+.030	—.075	0.05
400,400	.2275	.0145	+.130	+.035	—.080	—.005
425,100	.2360	.0195	+.135	+.035	—.080	—.010
449,800	.2470	.0227	+.145	+.035	—.085	—.010
474,500	.2595	.0262	+.150	+.035	—.100	—.010
500,500	.2710	.0290	+.155	+.035	—.100	—.010
525,200	.2840	.0380	+.160	+.035	—.105	—.010
549,900	.2988	.0455	+.165	+.040	—.110	—.010
574,600	.3135	.0552	+.170	+.040	—.110	—.010
600,600	.3305	.0647	+.175	+.040	—.120	—.020
650,000	.3795	.0920	+.180	+.030	—.150	—.020
699,400	.4395	.1585	+.180	+.015	—.170	—.030
750,100	.5210	.2420	+.170	+.010	—.200	—.030
799,500	.6295	.3320	+.195	+.025	—.280	—.040
835,900	.8370	1.2550	+.395	...	—.390	...

Failed by webs shearing rivets through ends of flange-angles and buckling at ends of pin-plates at plunger end. At tail-block end some rivets through webs and angles were loosened.

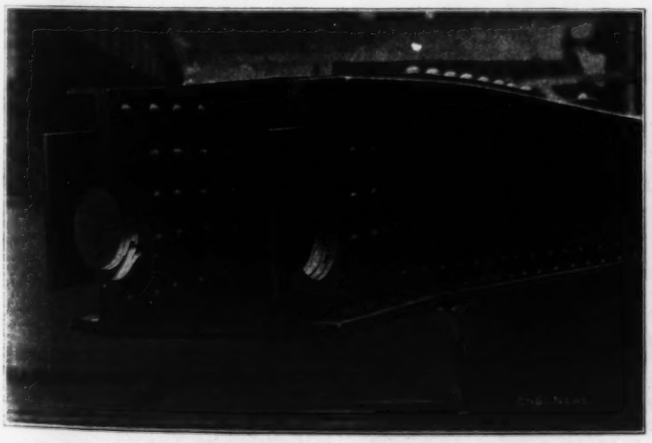


FIG. 5B. PLUNGER END AFTER FAILURE; WROUGHT-IRON TOP CHORD, BRIDGE 75, CINCINNATI DIVISION, P., C., C. & ST. L. RY.
Maximum load, 23,290 lbs. per sq. in.

FIG. 6B. PLUNGER END AFTER FAILURE; WROUGHT-IRON TOP CHORD, BRIDGE 25, C., R. & C. R. R.
Maximum load, 28,220 lbs. per sq. in.

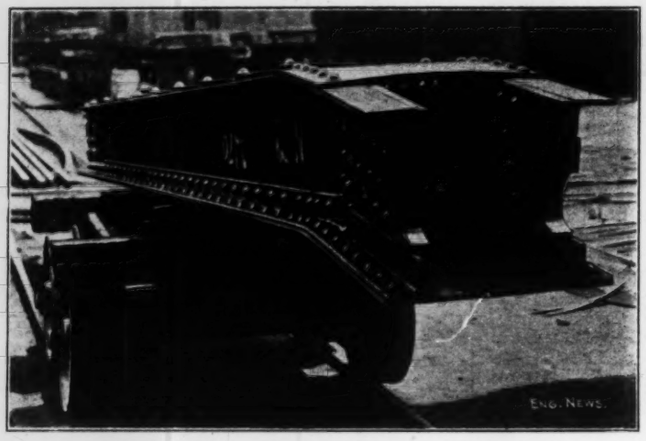


FIG. 7B. PLUNGER END AFTER FAILURE; WROUGHT-IRON TOP CHORD, BRIDGE 1, T., W. V. & O. R. R.
Maximum load, 29,210 lbs. per sq. in.

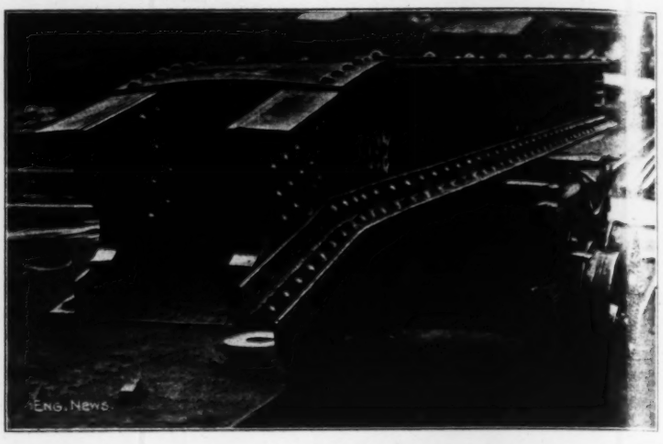


FIG. 8B. PLUNGER END AFTER FAILURE; WROUGHT-IRON TOP CHORD, BRIDGE 3, T., W. V. & O. R. R.
Maximum load, 29,900 lbs. per sq. in.

Fig. 9b. View After Failure; Steel Post, Bridge 17, T., W. V. & O. R. R.
Maximum load, 27,790 lbs. per sq. in.

Elastic limit 15,200 lbs. per sq. in. Maximum strength 29,900 lbs. per sq. in. The compression and the deflections, plotted on load, are shown in Fig. 8a.

The appearance of the member after failure is shown by Fig. 8b.

IX. STEEL POST, BRIDGE 17, T., W. V. & O. R. R. Tested May 17, 1892.

Material, Bessemer steel, el. lim. 39,000, ult. strength 62,000-65,600 lbs. per sq. in. Post built as per Fig. 9.

$A = 14.22$ sq. ins.; $r = 3.1$ ins. (about pin); $l/r = 97$.

Loads.	Compression in Inches.		Deflection in Inches.			
			Vertical.		Horizontal.	
	Load.	Set.	Load.	Set.	Load.	Set.
24,960	.0200	0.	+.04	0.	—.01	0.
50,180	.0451	0.	+.05	+.01	—.01	0.
74,880	.0632	0.	+.06	+.01	—.01	0.
100,100	.0830	0.	+.06	0.	—.01	+.01
119,600	.0995	—.0023	+.08	+.01	—.025	+.005
140,400	.1176	.0020	+.09	+.01	—.03	+.005
159,900	.1368	.0054	+.10	+.01	—.04	+.005
179,400	.1547	.0143	+.11	+.01	—.055	+.005
200,200	.1778	.0176	+.12	+.01	—.09	+.005
230,100	.2168	.0313	+.13	+.015	—.10	+.005
249,600	.2418	.0379	+.145	+.02	—.125	+.005
275,600	.2786	.0618	+.15	+.02	—.18	+.005
300,300	.3268	.0818	+.16	+.02	—.22	+.005
325,000	.3743	.1110	+.20	+.02	—.30	+.005
349,700	.4333	.1468	+.25	+.03	—.40	0.
374,400	.5178	.2068	+.30	+.05	—.68	—.03
395,200	.5968	.3773	+.33	+.05	...	...

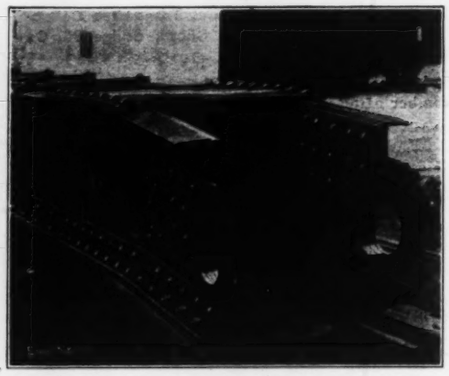


Fig. 10b. Tail-Block End After Failure; Wrought-Iron Top Chord, Bridge 18, T., W. V. & O. R. R.
Maximum load, 26,200 lbs. per sq. in.

Failed near center, the outstanding legs of angles buckling. The column as a whole, and the unsupported legs of the angles, were not absolutely straight before testing.

Elastic limit 12,600 lbs. per sq. in. Maximum strength 27,790 lbs. per sq. in. The compression and the deflections, plotted on load, are shown in Fig. 9a. The appearance of the member after failure is shown by Fig. 9b.

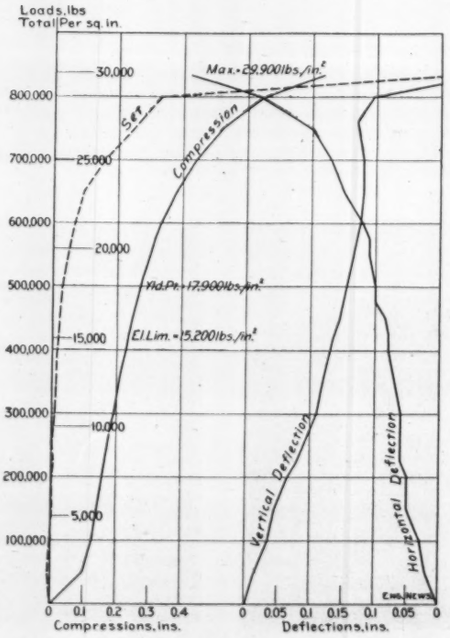


Fig. 8a. Compression, Set and Transverse Deflections; Wrought-Iron Top Chord, Bridge 3, T., W. V. & O. Ry.

X. WROUGHT-IRON TOP CHORD, BRIDGE 18, T., W. V. & O. R. R. Tested May 17, 1892.

Material wrought iron, el. lim. 30,500-32,960, ult. strength 48,340-52,740 lbs. per sq. in. Chord built as per Fig. 10.

$A = 28.38$ sq. ins.; $r = 6.34$ ins. (about pin); $l/r = 46$.

Loads.	Compression in Inches.		Deflection in Inches.			
			Vertical.		Horizontal.	
	Load.	Set.	Load.	Set.	Load.	Set.
49,400	.0420	.0004	0.	+.02	0.	0
100,100	.0597	—.0015	+.03	+.01	—.01	—.01
149,500	.0780	.0000	+.03	+.01	—.025	—.01
200,200	.0990	—.0054	+.05	+.01	—.035	—.01
224,900	.1067	—.0018	+.06	+.015	—.04	—.01
249,600	.1150	0.	+.065	+.015	—.04	—.01
275,600	.1292	—.0023	+.07	+.015	—.05	—.01
300,300	.1355	—.0016	+.07	+.015	—.05	—.01
325,000	.1455	.0001	+.08	+.015	—.05	—.01
349,700	.1530	.0005	+.08	+.01	—.055	—.01
374,400	.1648	.0030	+.09	+.01	—.06	—.01
400,400	.1765	.0017	+.10	+.01	—.07	—.02
425,100	.1860	.0107	+.10	+.01	—.075	—.02
449,800	.1975	.0177	+.105	+.01	—.08	—.02
474,500	.2065	.0248	+.11	+.01	—.08	—.02
500,500	.2205	.0368	+.12	+.01	—.09	—.02
525,200	.2330	.0505	+.125	0.	—.09	—.02
549,900	.2475	.0567	+.13	0.	—.09	—.02
574,600	.2627	.0643	+.13	—.01	—.09	—.03
600,600	.2795	.0726	+.135	—.015	—.09	—.03
650,000	.3355	.1019	+.16	—.01	—.09	—.03
699,400	.4205	.1641	+.215	+.01	—.11	—.03
743,000	.5605	.1981	+.34	+.145	—.15	—.09

Failed by webs shearing rivets through ends of flange-angles and buckling at ends of pin-plates at tail-block end.

Elastic limit 16,700 lbs. per sq. in. Maximum strength 26,200 lbs. per sq. in. The compressions and deflections, plotted on load, are shown in Fig. 10a. The appearance of the member after failure is shown by Fig. 10b.

XI. WROUGHT-IRON TOP CHORD, BRIDGE 18, T., W. V. & O. R. R. SECOND TEST-SECTION. Tested May 18, 1892.

Material, wrought iron, el. lim. 30,500-32,960, ult. strength 48,340-52,740 lbs. per sq. in. Chord built as per Fig. 11.

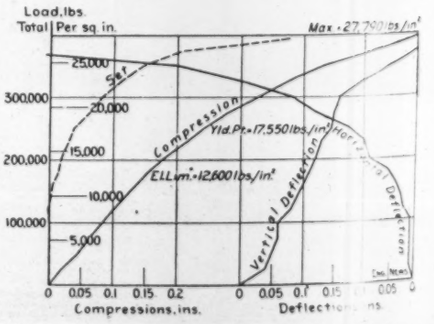


Fig. 9a. Compression, Set and Transverse Deflections; Steel Intermediate Post, Bridge 17, T., W. V. & O. Ry.

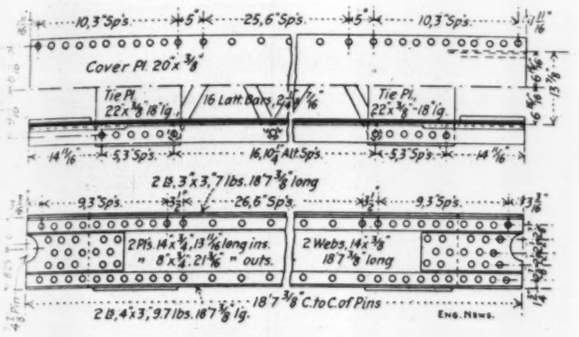

Cover Pl. 20"x 3/8"
Tie Pl. 22"x 3/8" 18'lg.
16 Latt Bars, 2 1/4"x 7/16"
Tie Pl. 22"x 3/8"-18'lg.
16,10 1/4" Alt. Sp's.
2 Pls. 14"x 3/8", 13 7/16" long ins.
2 Webs, 14"x 3/8" 18'7 3/8" long
18'7 3/8" C. to C. of Pins
ENG. NEWS.

FIG. 7. TOP CHORD, BRIDGE 1, T., W. V. & O. R. R.

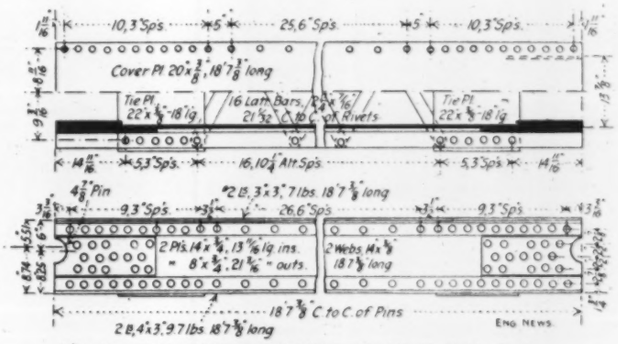

Cover Pl. 20"x 3/8", 18'7 3/8" long
Tie Pl. 22"x 3/8"-18'lg.
16 Latt Bars, 2 1/4"x 7/16"
Tie Pl. 22"x 3/8"-18'lg.
16,10 1/4" Alt. Sp's.
2 Pls. 14"x 3/8", 13 7/16" lg. ins.
2 Webs, 14"x 3/8" 18'7 3/8" long
18'7 3/8" C. to C. of Pins
ENG. NEWS.

FIG. 8. TOP CHORD, BRIDGE 3, T., W. V. & O. R. R.

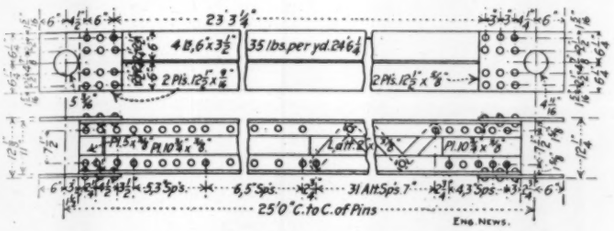


FIG. 9. INTERMEDIATE POST, BRIDGE 17, T., W. V. & O. R. R.

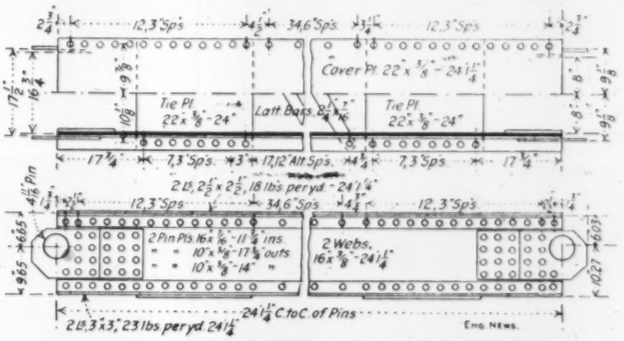

Cover Pl. 22"x 3/8"-24 1/4"
Tie Pl. 22"x 3/8"-24"
Latt Bars, 2 1/4"x 7/16"
Tie Pl. 22"x 3/8"-24"
2 Pin Pls. 16"x 3/8"-11 1/4" ins.
2 Webs, 16"x 3/8"-24 1/4"
24 1/4" C. to C. of Pins
ENG. NEWS.

FIG. 10. TOP CHORD, BRIDGE 18, T., W. V. & O. R. R.

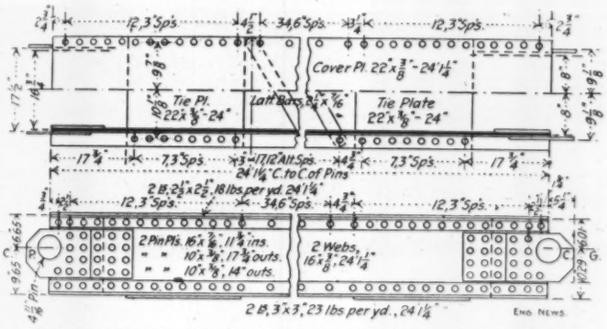

Cover Pl. 22"x 3/8"-24 1/4"
Tie Pl. 22"x 3/8"-24"
Latt Bars, 2 1/4"x 7/16"
Tie Plate 22"x 3/8"-24"
24 1/4" C. to C. of Pins
2 Pin Pls. 16"x 3/8"-11 1/4" ins.
2 Webs, 16"x 3/8"-24 1/4"
ENG. NEWS.

FIG. 11. SECOND TEST CHORD, BRIDGE 18, T., W. V. & O. R. R.

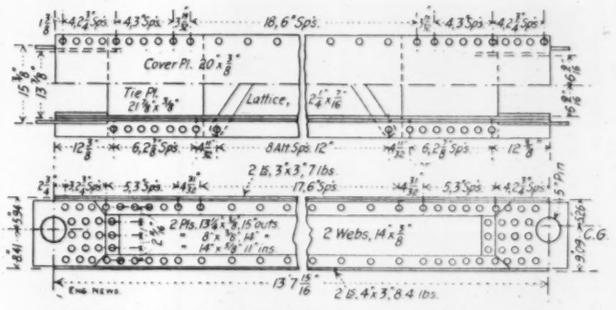


FIG. 12. TOP CHORD, BRIDGE 21, T., W. V. & O. R. R.

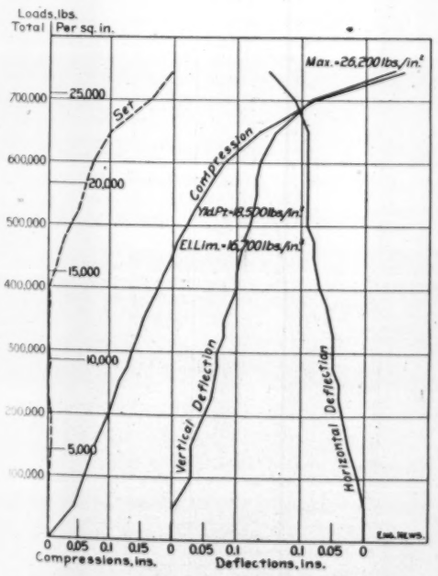


Fig. 10a. Compression, Set and Transverse Deflections; Second Test, Wrought-Iron Top Chord, Bridge 18, T., W. V. & O. Ry.

$A = 28.20$ sq. ins.; $r = 6.33$ ins. (about pin); $l/r = 46$.

Loads.	Compression in Inches.		Deflection in Inches.			
			Vertical.		Horizontal.	
	Load.	Set.	Load.	Set.	Load.	Set.
49,400	.0375	0.	0.	0.	0.	0.
100,100	.0529	0.	0.	0.	0.	0.
149,500	.0745	—.0013	0.	0.	—.01	0.
200,200	.0937	—.0009	0.	0.	—.01	—.005
224,900	.1033	.0015	0.	0.	—.02	—.005
249,600	.1107	.0012	—.01	0.	—.02	—.01
275,600	.1254	0.	+.01	0.	—.025	—.01
300,300	.1300	.0003	+.05	0.	—.025	—.01
325,000	.1425	.0007	+.09	0.	—.025	—.01
349,700	.1535	.0044	+.10	0.	—.026	—.01
374,400	.1625	.0023	+.11	+.005	—.03	—.01
400,400	.1745	.0042	+.11	+.005	—.04	—.01
425,100	.1837	.0064	+.12	+.01	—.04	—.01
449,800	.1945	.0079	+.13	+.015	—.05	—.01
474,500	.2070	.0144	+.135	+.015	—.05	—.01
500,500	.2225	.0225	+.14	+.015	—.06	—.02
525,200	.2370	.0338	+.15	+.015	—.065	—.02
549,900	.2475	.0445	+.16	+.015	—.065	—.02
574,600	.2662	.0582	+.17	+.02	—.065	—.02
600,600	.2833	.0667	+.19	+.02	—.07	—.02
650,000	.3305	.0894	+.22	+.03	—.07	—.02
699,400	.4075	.1505	+.27	+.06	—.09	—.03
739,700	.5625	.1920	+.50	+.29	—.14	...

Failed by webs shearing rivets through ends of flange-angles and buckling at ends of pin-plates at tail-block end.

Elastic limit 15,950 lbs. per sq. in. Maximum strength 26,230 lbs. per sq. in. The compression and deflections, plotted on load, are shown in Fig. 11a. The appearance of the member after failure is shown by Fig. 11b.

XII. WROUGHT-IRON TOP CHORD, BRIDGE 21, T., W. V. & O. R. R. Tested Feb. 17, 1893.

Material wrought iron, el. lim. 30-200-31,410, ult. strength 50,380-51,700 lbs. per sq. in. Chord built as per Fig. 12.

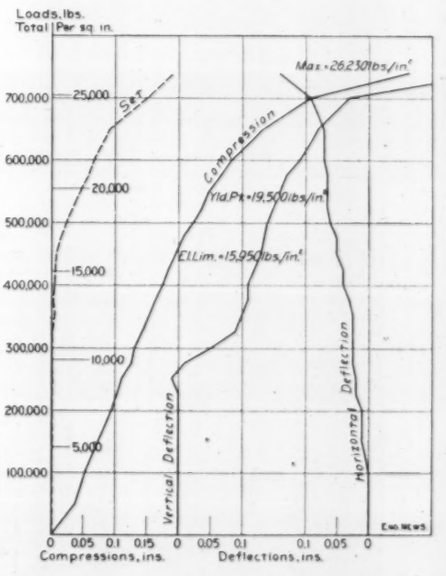
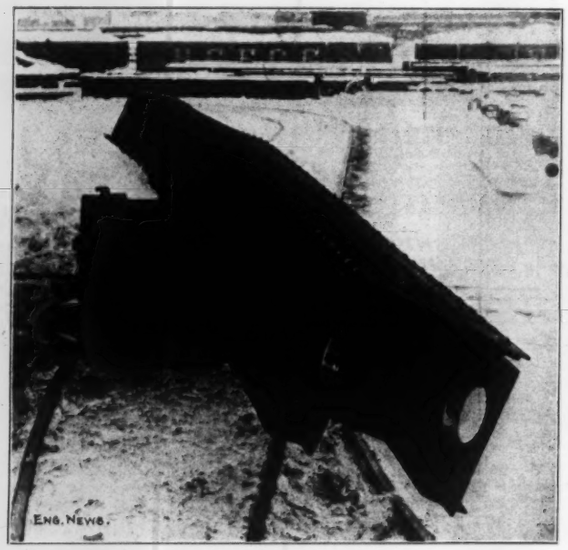


Fig. 11a. Compression, Set and Transverse Deflections; First Test, Wrought-Iron Top Chord, Bridge 18, T., W. V. & O. Ry.

FIG. 11B. TAIL-BLOCK END AFTER FAILURE; SECOND TEST CHORD, BRIDGE 18, T., W. V. & O. R. R.
Maximum load, 26,230 lbs. per sq. in.

FIG. 12B. PLUNGER END AFTER FAILURE; WROUGHT-IRON TOP CHORD, BRIDGE 21, T., W. V. & O. R. R.
Maximum load, 30,770 lbs. per sq. in.



FIGS. 13B AND 13C. RIGHT AND LEFT SIDES AFTER FAILURE; WROUGHT-IRON TOP CHORD, BRIDGE 24, T., W. V. & O. R. R.
Maximum load, 28,200 lbs. per sq. in.

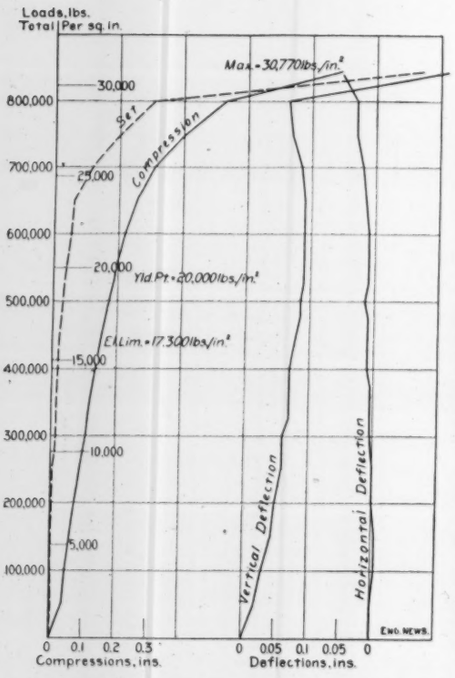


Fig. 12a. Compression, Set and Transverse Deflections; Wrought-Iron Top Chord, Bridge 21, T., W. V. & O. Ry.

$A = 27.46$ sq. ins.; $r = 5.65$ ins. (about pin); $l/r = 29$.

Loads.	Compression in Inches.		Deflection in Inches.			
			Vertical.		Horizontal.	
	Load.	Set.	Load.	Set.	Load.	Set.
49,400	.0402	.0044	+.02	—.01	...	0.
100,100	.0484	.0025	+.03	0.	+.005	+.005
149,500	.0630	.0029	+.045	+.01	+.005	0.
200,200	.0758	.0038	+.05	+.015	0.	0.
224,900	.0844	.0041	+.055	+.02	0.	0.
249,600	.0908	.0050	+.06	+.02	0.	0.
275,600	.1001	.0122	+.06	+.02	—.003	—.003
300,300	.1085	.0133	+.06	+.02	—.005	—.005
325,000	.1133	.0138	+.07	+.02	—.005	—.005
349,700	.1206	.0158	+.07	+.02	—.005	—.005
374,400	.1279	.0163	+.07	+.025	—.005	—.005
400,400	.1363	.0186	+.07	+.025	—.01	—.01
425,100	.1453	.0188	+.075	+.03	—.01	—.005
449,800	.1543	.0224	+.08	+.03	—.01	—.005
474,500	.1633	.0254	+.085	+.03	—.01	—.005
500,500	.1733	.0289	+.085	+.03	—.015	—.01
525,200	.1858	.0333	+.09	+.03	—.01	—.008
549,900	.1945	.0396	+.09	+.03	—.01	—.01
574,600	.2069	.0451	+.09	+.03	—.01	—.008
600,600	.2203	.0529	+.09	+.03	—.01	—.008
650,000	.2558	.0591	+.09	+.025	—.015	—.01
699,400	.3108	.1180	+.085	+.015	—.02	—.015
750,100	.4033	.2026	+.07	—.01	—.03	—.02
799,500	.5308	.3083	+.065	—.02	—.03	—.02
845,000	.8858	1.1463	+.32	—.31	—.045	—.01

Failed at one end, vertical legs of top angles tearing through rivet-hole at end of pin-plates and along fillet. Webs buckled in front of pin-plates.

Elastic limit 17,300 lbs. per sq. in. Maximum strength 30,770 lbs. per sq. in. The compression and deflections, plotted on load, are shown in Fig. 12a. The appearance of the member after failure is shown by Fig. 12b.

XIII. WROUGHT-IRON TOP CHORD, BRIDGE 24, T., W. V. & O. R. R. Tested Feb. 17, 1893.

Material wrought iron, el. lim. 30,070-31,100, ult. strength 48,400-51,910 lbs. per sq. in. Chord built as per Fig. 13.

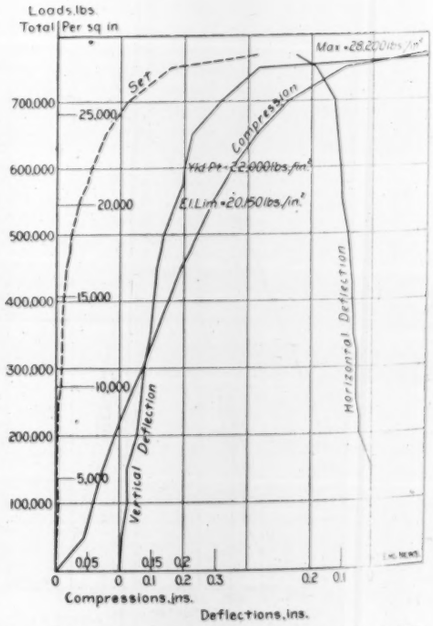


Fig. 13a. Compression, Set and Transverse Deflections; Wrought-Iron Top Chord, Bridge 24, T., W. V. & O. Ry.

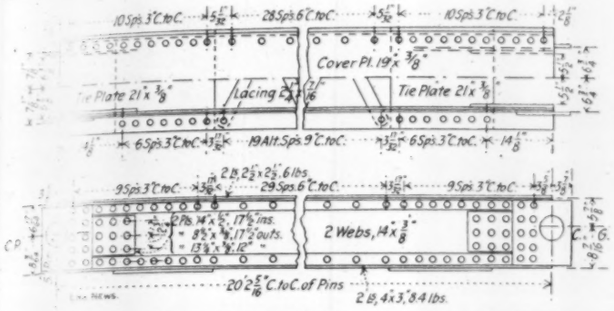


FIG. 13. TOP CHORD, BRIDGE 24, T., W. V. & O. R. R.

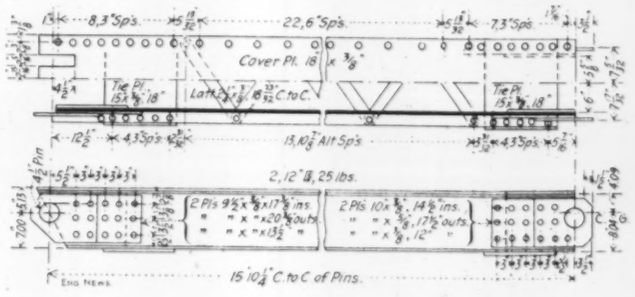


FIG. 14. TOP CHORD, BRIDGE 31, C. & M. V. RY.

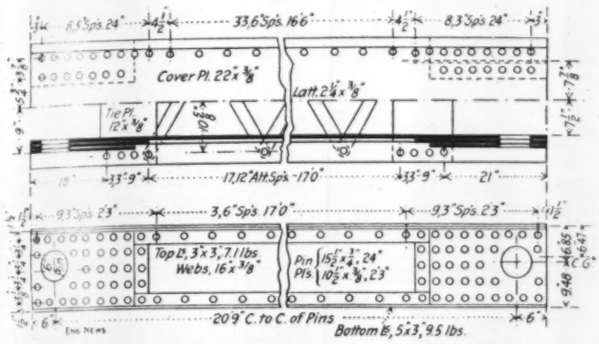


FIG. 15. TOP CHORD, BRIDGE 104, C. & M. V. RY.

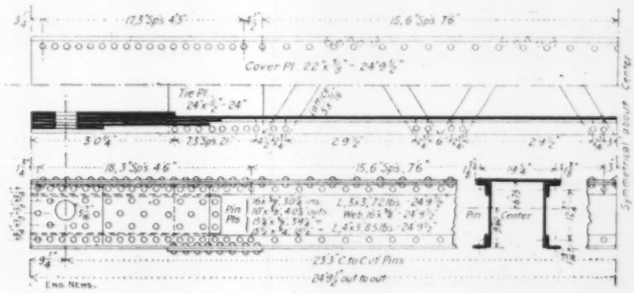


FIG. 16. TOP CHORD, SPAN C, NEWPORT & CINCINNATI BRIDGE.

$A = 27.28$ sq. ins.; $r = 5.8$ ins. (about pin); $l/r = 42$.

Loads.	Compression in Inches. Load.	Set.	Deflection in Inches. Vertical. Load.	Set.	Horizontal. Load.	Set.
49,400	.0447	—.0000	+.005	0.	0.	0.
100,100	.0595	—.0023	+.025	+.005	0.	0.
149,500	.0737	—.0011	+.025	+.005	0.	0.
200,200	.0923	.0027	+.055	+.005	—.04	—.01
224,900	.1020	.0029	+.06	+.005	—.04	—.01
249,600	.1125	.0035	+.065	+.01	—.05	—.01
275,600	.1239	.0073	+.07	+.015	—.05	—.01
300,300	.1360	.0093	+.075	+.025	—.05	—.01
325,000	.1455	.0089	+.085	+.025	—.05	—.01
349,700	.1547	.0095	+.095	+.025	—.06	—.01
374,400	.1649	.0102	+.10	+.025	—.06	—.01
400,400	.1762	.0112	+.105	+.025	—.065	—.01
425,100	.1852	.0116	+.11	+.025	—.065	—.01
449,800	.1961	.0143	+.115	+.025	—.07	—.02
474,500	.2096	.0204	+.125	+.025	—.075	—.02
500,500	.2202	.0225	+.135	+.025	—.08	—.025
525,200	.2337	.0282	+.155	+.03	—.09	—.03
549,900	.2466	.0357	+.175	+.035	—.10	—.03
574,600	.2622	.0481	+.195	+.035	—.10	—.03
600,600	.2764	.0557	+.20	+.045	—.105	—.035
650,000	.3152	.0764	+.225	+.065	—.115	—.04
699,400	.3645	.1098	+.315	+.105	—.125	—.06
750,100	.4567	.1786	+.435	+.225	—.185	—.10
769,600	.5962	.3166	+1.235	+.855	—.25	—.165

Failed by crippling of lower flanges near center of chord. Web-plates showed slight inward deflection at one end near the pin-plates.

Elastic limit 20,150 lbs. per sq. in. Maximum strength 28,200 lbs. per sq. in. The compression and deflections, plotted on load, are shown in Fig. 13a. The appearance of the two sides of the chord after failure is shown by Figs. 13b and 13c.

XIV. WROUGHT-IRON TOP CHORD, BRIDGE 31, C. & M. V. RY. Tested Feb. 18, 1893.

Material wrought iron, el. lim. 29,650-31,440, ult. strength 50,000-52,630 lbs. per sq. in. Chord built as per Fig. 14.

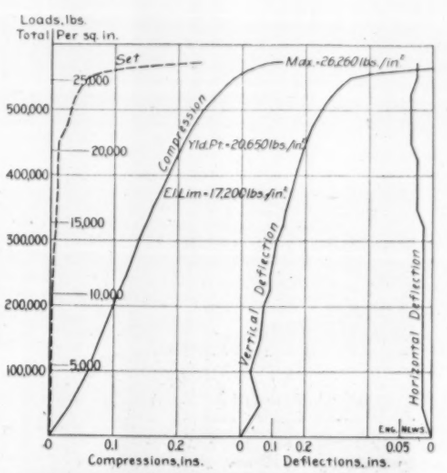


Fig. 14a. Compression, Set and Transverse Deflections; Wrought-Iron Top Chord, Bridge 31, C. & M. V. Ry.

$A = 21.78$ sq. ins.; $r = 4.71$ ins. (about pin); $l/r = 40$.

Neutral axis about 0.47-in. above center of pin.

FIG. 14B. RIGHT SIDE AFTER FAILURE; WROUGHT-IRON TOP CHORD, BRIDGE 31, C. & M. V. RY.
Maximum load, 26,260 lbs. per sq. in.

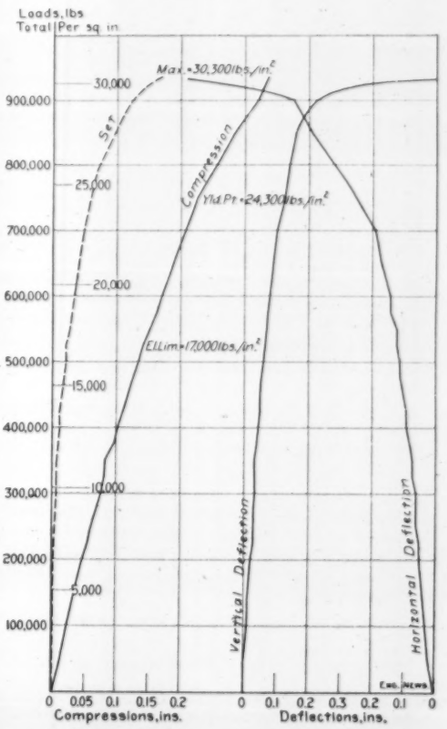


Fig. 15a. Compression, Set and Transverse Deflections; Wrought-Iron Top Chord, Bridge 104, C. & M. V. Ry.

Loads.	Compression in Inches. Load.	Set.	Deflection in Inches. Vertical. Load.	Set.	Horizontal. Load.	Set.
49,400	.0337	.0015	+.06	+.005	—.015	—.005
100,100	.0605	.0020	+.03	+.01	—.015	—.005
149,500	.0791	.0020	+.06	+.01	—.015	—.005
200,200	.0993	.0029	+.07	+.01	—.015	—.005
224,900	.1099	.0031	+.09	+.01	—.015	—.005
249,600	.1203	.0039	+.09	+.005	—.015	—.005
275,600	.1288	.0045	+.10	+.005	—.015	—.015
300,300	.1390	.0070	+.11	+.005	—.015	—.015
325,000	.1505	.0082	+.13	+.01	—.015	—.015
349,700	.1607	.0095	+.14	+.01	—.025	—.015
374,400	.1724	.0115	+.155	+.02	—.025	—.005
400,400	.1845	.0142	+.17	+.015	—.025	—.005
425,100	.1970	.0168	+.185	+.02	—.025	—.005
449,800	.2087	.0198	+.20	+.03	—.035	—.005
474,500	.2232	.0250	+.22	+.04	—.035	—.005
505,500	.2418	.0320	+.25	+.05	—.035	—.005
525,200	.2628	.0430	+.285	+.07	—.035	—.005
549,900	.2880	.0604	+.345	+.095	—.025	+.005
572,600	.3635	.1370	+.76	+.43	—.025	+.005

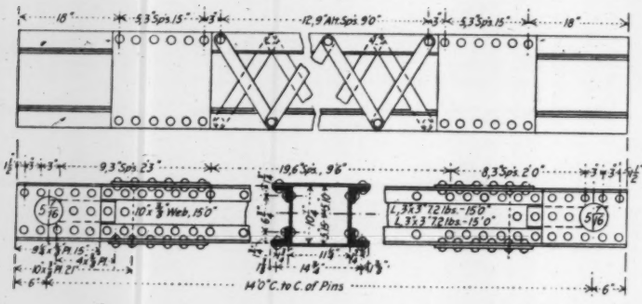


FIG. 17. INTERMEDIATE POST, SPAN C, NEWPORT & CINCINNATI BRIDGE.

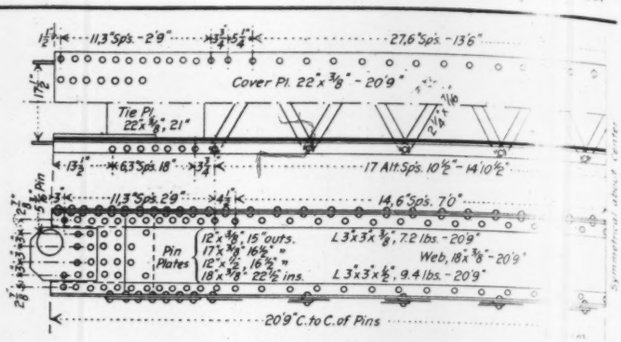


FIG. 18. TOP CHORD, BRIDGE 20, LOUISVILLE DIVISION, C., C. & ST. L. RY.

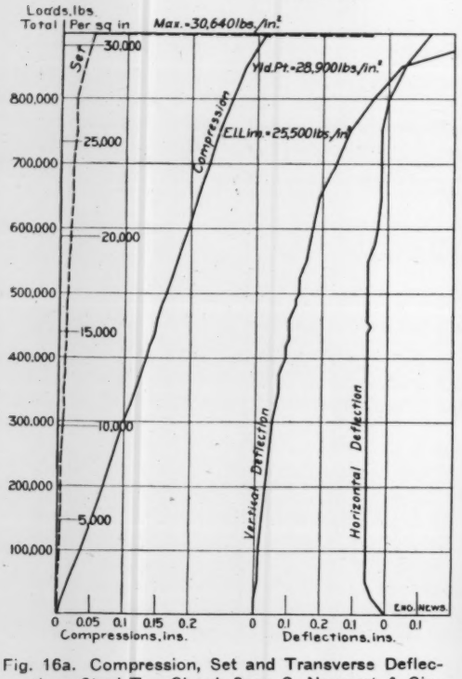


Fig. 16a. Compression, Set and Transverse Deflections Steel Top Chord, Span C, Newport & Cincinnati Bridge.

Failed by crippling of lower flanges near center of column.

Elastic limit 17,200 lbs. per sq. in. Maximum strength 26,260 lbs. per sq. in. The compression and deflections, plotted on load, are shown in Fig. 14a. A view of the right-hand side of the chord after failure is given by Fig. 14b.

XV. WROUGHT-IRON TOP CHORD, BRIDGE 104, C. & M. V. RY. Tested Dec. 12, 1893.

Material wrought iron, el. lim. 30,230-32,250, ult. strength 49,100-51,240 lbs. per sq. in. Chord built as per Fig. 15.

$A = 30.81$ sq. ins.; $r = 6.51$ ins. (about pin); $l/r = 37$.

Neutral axis 0.14-in. below center of pin.

Loads.	Compression in Inches.		Deflection in Inches.			
			Vertical.		Horizontal.	
	Load.	Set.	Load.	Set.	Load.	Set.
49,400	.0125	.0	0.	0.	—.02	—.01
100,100	.0230	.0003	+.010	0.	—.03	—.01
149,500	.0347	.0003	+.015	0.	—.04	—.01
200,000	.0475	.0019	+.020	0.	—.05	—.01
224,900	.0548	.0028	+.025	0.	—.05	—.01
249,600	.0612	.0038	+.030	+.005	—.06	—.01
275,600	.0685	.0050	+.030	+.005	—.06	—.01
300,300	.0760	.0060	+.035	+.005	—.06	—.01
325,000	.0810	.0063	+.035	+.005	—.07	—.01
349,700	.0830	.0064	+.035	+.005	—.07	—.01
374,400	.0973	.0089	+.040	+.005	—.08	—.01
400,400	.1039	.0113	+.045	+.005	—.09	—.01
425,100	.1107	.0130	+.050	+.005	—.09	—.01
449,800	.1185	.0152	+.050	+.005	—.10	—.01
474,500	.1275	.0178	+.055	+.005	—.11	—.015
500,500	.1343	.0206	+.060	+.005	—.11	—.02
525,200	.1415	.0239	+.065	+.005	—.12	—.02
549,900	.1522	.0273	+.070	+.005	—.12	—.02
574,600	.1625	.0304	+.075	+.010	—.14	—.02
600,500	.1707	.0336	+.080	+.010	—.14	—.02
650,000	.1885	.0404	+.090	+.010	—.17	—.03
699,400	.2080	.0482	+.105	+.015	—.19	—.03
750,100	.2232	.0588	+.125	+.020	—.25	—.04
799,500	.2535	.0729	+.140	+.030	—.32	—.05
850,200	.2850	.0977	+.160	+.050	—.39	—.10
899,600	.3205	.1239	+.225	+.090	—.45	—.12
933,400	.3370	.1710	+1.055	+.925	—.79	—.51

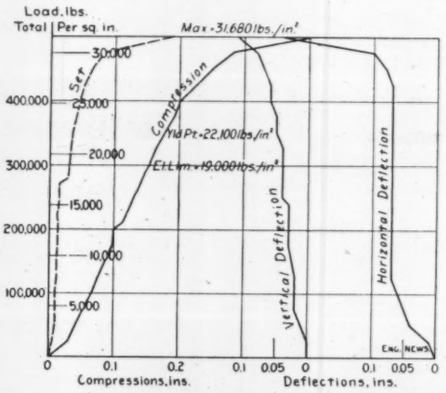


Fig. 17a. Compression, Set and Transverse Deflections; Steel Intermediate Post, Span C, Newport & Cincinnati Bridge.

Failed by crippling of lower flanges in end quarter.

Elastic limit 17,000 lbs. per sq. in. Maximum strength 30,300 lbs. per sq. in. The compression and deflections, plotted on load, are shown in Fig. 15a. The end of the chord near which the crippling was localized is shown by Fig. 15b.

XVI. STEEL TOP CHORD, SPAN C, NEWPORT & CINCINNATI BRIDGE. Tested July 21, 1897.

Material open-hearth steel about 0.21 carbon, ultimate strength 63,000-66,000 lbs. per sq. in. Chord built as per Fig. 16.

$A = 29.36$ sq. ins.; $r = 6.44$ ins. (about pin); $l/r = 46$.

Neutral axis 0.02-in. above pin center.

Loads.	Compression in Inches.		Deflection in Inches.			
			Vertical.		Horizontal.	
	Load.	Set.	Load.	Set.	Load.	Set.
24,900	.0078	.0007	—.04	0.	0.	0.
49,400	.0164	.0018	—.06	0.	+.01	0.
100,100	.0349	.0029	—.06	0.	+.01	0.
149,500	.0529	.0040	—.06	0.	+.02	+.01
200,200	.0684	.0043	—.06	0.	+.03	+.01
249,600	.0838	.0047	—.06	0.	+.04	+.01
300,300	.0994	.0069	—.06	—.01	+.05	+.01
325,000	.1087	.0084	—.06	—.02	+.06	+.01
349,700	.1172	.0077	—.06	—.02	+.07	+.02
374,400	.1244	.0093	—.06	—.02	+.07	+.02
400,400	.1333	.0086	—.06	—.02	+.09	+.02
409,500	.1347	.0097	—.06	—.04	+.09	+.02
419,900	.1389	.0102	—.06	—.02	+.09	+.02
430,300	.1420	.0103	—.06	—.03	+.10	+.02
439,400	.1447	.0103	—.06	—.02	+.10	+.02

FIG. 15B. LEFT SIDE, TAIL-BLOCK END, AFTER FAILURE; WROUGHT-IRON TOP CHORD, BRIDGE 104, C. & M. V. RY.
Maximum load, 30,300 lbs. per sq. in.

FIG. 16B. LEFT SIDE AFTER FAILURE; STEEL TOP CHORD, SPAN C, OF NEWPORT & CINCINNATI BRIDGE.
Maximum load, 30,640 lbs. per sq. in.

FIG. 17B. VIEW AFTER FAILURE; STEEL POST, SPAN C, OF NEWPORT & CINCINNATI BRIDGE.
Maximum load, 31,080 lbs. per sq. in

FIG. 18B. RIGHT SIDE AFTER FAILURE; FIRST TEST, STEEL TOP CHORD, BRIDGE 20, LOUISVILLE DIVISION, P., C., C. & ST. L. RY.
Maximum load, 31,980 lbs. per sq. in.

449,800	.1470	.0110	—.05	—.02	+.10	+.02
460,200	.1492	.0109	—.06	—.02	+.10	+.03
470,600	.1520	.0114	—.06	—.03	+.11	+.03
479,700	.1547	.0109	—.06	—.03	+.12	+.03
490,100	.1584	.0116	—.06	—.03	+.12	+.03
500,500	.1631	.0125	—.06	—.03	+.13	+.035
525,200	.1701	.0133	—.06	—.03	+.13	+.035
549,900	.1795	.0136	—.06	—.03	+.15	+.035
574,600	.1872	.0155	—.04	—.03	+.16	+.04
600,600	.1959	.0158	—.03	—.03	+.17	+.04
650,000	.2102	.0192	—.02	—.04	+.19	+.05
699,400	.2272	.0190	—.02	—.03	+.24	+.06
750,100	.2422	.0241	—.02	—.03	+.28	+.08
799,500	.2610	.0228	0.	—.01	+.35	+.10
850,200	.2811	.0344	+.05	+.01	+.44	+.16
899,600	.3141	.0491	+.13	+.01	+.77	+.36
897,000	.3172	.4574	...	+2.02	+.89	...

Failed by crippling of lower flanges near center of chord.

Elastic limit 25,500 lbs. per sq. in. Maximum strength 30,640 lbs. per sq. in. The compression and deflections are shown in Fig. 16a. Fig. 16b gives a side view of the chord after failure.

XVII. STEEL INTERMEDIATE POST, SPAN C, NEWPORT & CINCINNATI BRIDGE. Tested July 21, 1897.

Material open-hearth steel, 0.21-0.24 carbon, ultmiate strength 63,000-68,000 lbs. per sq. in. Post built as per Fig. 17.

$A = 15.8$ sq. ins.; $r = 3.72$ ins. (about pin); $l/r = 45$.

Post was not in perfect alinement as it came from shop.

Loads.	Compression in Inches. Load.	Set.	Deflection in Inches. Vertical. Load.	Set.	Horizontal. Load.	Set.
24,960	.0300	.0063	0.	0.	—.01	—.01
49,400	.0419	.0104	—.01	0.	—.04	—.02
75,400	.0541	.0113	—.02	0.	—.05	—.03
100,100	.0665	.0096	—.02	—.01	—.06	—.03
124,800	.0794	.0120	—.02	—.01	—.07	—.03
149,500	.0879	.0121	—.025	—.01	—.07	—.04
175,500	.0986	.0132	—.03	—.01	—.07	—.04
200,200	.1084	.0138	—.03	—.01	—.07	—.04
210,600	.1129	.0142	—.03	—.01	—.07	—.04
219,700	.1179	.0121	—.03	—.01	—.07	—.04
230,100	.1224	.0143	—.03	—.01	—.07	—.04
240,500	.1268	.0142	—.03	—.01	—.07	—.04
249,600	.1304	.0158	—.04	—.01	—.07	—.04
260,000	.1361	.0154	—.04	—.01	—.07	—.04
270,400	.1400	.0160	—.04	—.01	—.07	—.04
279,500	.1439	.0294	—.04	—.01	—.07	—.04
289,900	.1479	.0297	—.04	—.01	—.07	—.04
300,300	.1524	.0304	—.04	—.01	—.07	—.04
325,000	.1649	.0324	—.04	—.01	—.07	—.04
349,700	.1779	.0346	—.05	—.02	—.07	—.04
374,400	.1919	.0404	—.05	—.02	—.07	—.04
400,400	.2072	.0458	—.06	—.02	—.07	—.04
425,100	.2254	.0521	—.06	—.02	—.07	—.04
449,800	.2504	.0642	—.07	—.03	—.08	—.04
474,500	.2837	.0848	—.08	—.04	—.10	—.05
500,500	.4054	.1881	—.11	+.12	—.24	—.18

Failed by crippling of flanges near center.

Elastic limit 19,000 lbs. per sq. in. Maximum strength 31,680 lbs. per sq. in. The compression and deflections plotted on load, are shown in Fig. 17a. The appearance of the post after failure is shown by Fig. 17b.

XVIII. STEEL TOP CHORD, BRIDGE 20, LOUISVILLE DIVISION, P., C., C. & ST. L. RY. Tested April 11, 1900.

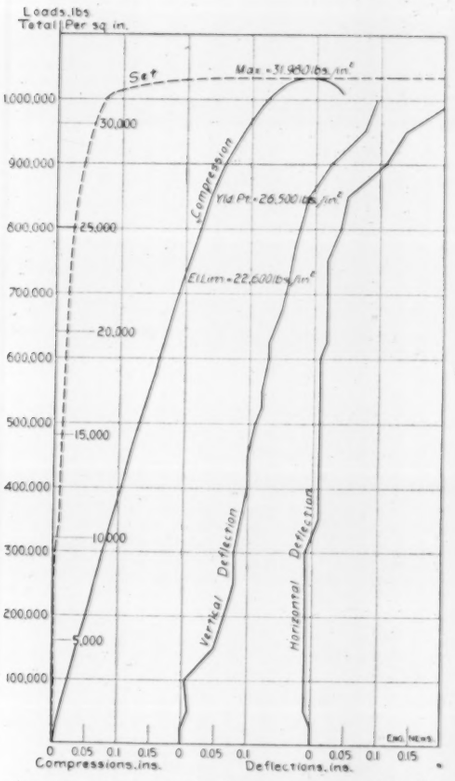


Fig. 18a. Compression, Set and Transverse Deflections; Test 1, Steel Top Chord, Bridge 20, Louisville Division, P., C., C. & St. L. Ry.

Material open-hearth steel, 0.14-0.17 carbon, ultimate strength 61,000-65,000 lbs. per sq. in. Chord built as per Fig. 18.

$A = 32.11$ sq. ins.; $r = 7.29$ ins. (about pin); $l/r = 34$.

Loads.	Compression in Inches. Load.	Set.	Deflection in Inches. Vertical. Load.	Set.	Horizontal. Load.	Set.
24,960	.0055	0.	0.	0.	0	0
49,400	.0105	.0005	+.01	0.	—.01	0
100,100	.0244	.0020	+.005	+.01	—.01	0
149,500	.0370	.0005	+.05	+.01	—.01	0
200,200	.0513	.0005	...	+.01	—.01	0
249,600	.0640	0.	+.08	+.01	—.01	0
300,300	.0775	.0030	+.08	+.01	—.01	0
349,700	.0910	.0100	+.09	+.01	+.01	0
400,400	.1060	.0112	+.10	+.015	+.01	0
449,800	.1175	.0117	+.10	+.02	+.01	0
500,500	.1313	.0125	+.11	+.02	+.01	0
525,200	.1392	.0135	+.12	+.02	+.01	0
549,900	.1465	.0145	+.12	+.015	+.01	0
574,600	.1544	.0155	+.125	+.02	+.01	0
600,600	.1608	.0163	+.13	+.02	+.01	0
625,300	.1680	.0173	+.13	+.02	+.02	0
650,000	.1755	.0184	+.14	+.025	+.02	0
674,700	.1823	.0193	+.15	+.025	+.02	0
699,400	.1900	.0201	+.155	+.025	+.02	0
725,400	.1974	.0211	+.16	+.025	+.02	0
750,100	.2065	.0230	+.165	+.03	+.02	0
774,800	.2150	.0255	+.17	+.03	+.03	0
799,500	.2230	.0275	+.177	+.035	+.04	+.01
850,200	.2397	.0330	+.19	+.05	+.04	+.01
899,600	.2905	.0419	+.225	+.06	+.11	+.01
950,300	.2900	.0545	+.275	+.08	+.14	+.03
999,700	.3252	.0755	+.295	+.15	+.22	+.03
1,027,000	...	.5915	...	+3.03	+.26	—.55

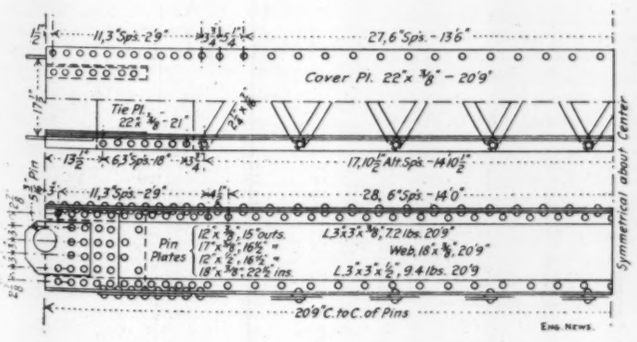

11,3"Spc.-2'9"
27,6"Spc.-13'6"
Cover Pl. 22"x 3/8" - 20'9"
Tie Pl. 22"x 3/8" - 21"
17,10 1/2" Alt. Spc. - 14 10 1/2"
28, 6"Spc. - 14'0"
Pin
Plates
12"x 3/8", 15" cuts.
17"x 3/8", 16 1/2" "
12"x 1/2", 16 1/2" "
18"x 3/8", 22 5/8" ins.
L 3"x 3"x 3/8", 7.2 lbs. 20'9"
Web, 18"x 3/8", 20'9"
L 3"x 3"x 1/2", 9.4 lbs. 20'9
Symmetrical about Center
20'9" C. to C. of Pins
Eng. News.

FIG. 19. SECOND TEST CHORD, BRIDGE 20, LOUISVILLE DIVISION, P., C., C. & ST. L. RY.

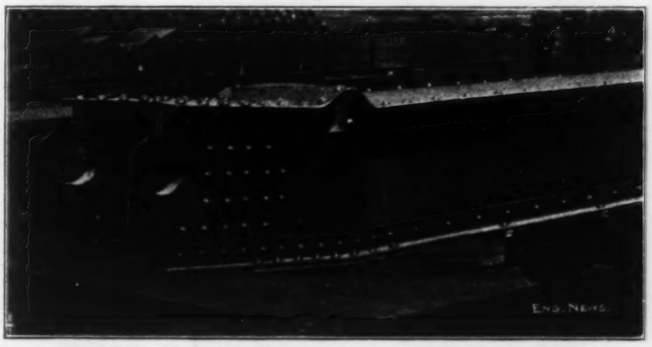


FIG. 19B. LEFT SIDE AFTER FAILURE; SECOND TEST, STEEL TOP CHORD, BRIDGE 20, LOUISVILLE DIVISION, P. C., C. & ST. L. RY.
Maximum load, 33,960 lbs. per sq. in.

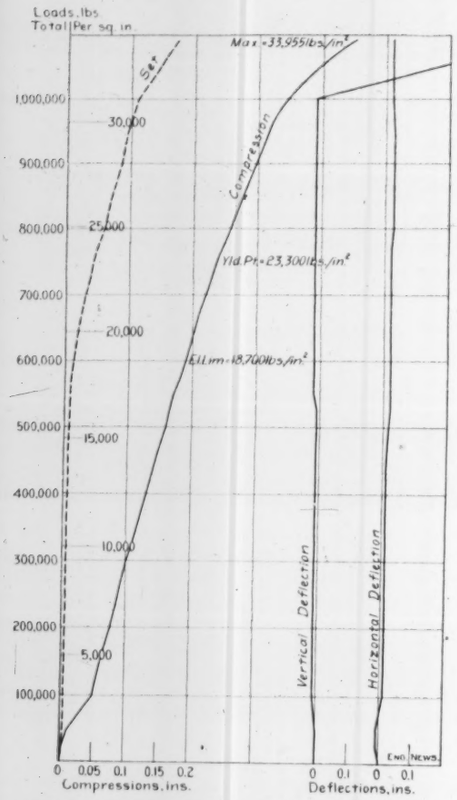


Fig. 19a. Compression, Set and Transverse Deflections; Test 2, Steel Top Chord, Bridge 20, Louisville Division, P., C., C. & St. L. Ry.

Failed by crippling of lower flanges near center.

Elastic limit 22,600 lbs. per sq. in. Maximum strength 31,980 lbs. per sq. in. The compression and deflections, plotted on load, are shown in Fig. 18a. A side view of the chord after failure is shown by Fig. 18b.

XIX. STEEL TOP CHORD, BRIDGE 20, LOUISVILLE DIVISION, P., C., C. & ST. L. RY. Tested April 11, 1900.

Material open-hearth steel, same as for XVIII. Chord built as per drawing Fig 19.

$A = 32.16$ sq. ins.; $r = 7.20$ ins. (about pin); $l/r = 35$.

Loads.	Compression in Inches.		Deflection in Inches.			
			Vertical.		Horizontal.	
	Load.	Set.	Load.	Set.	Load.	Set.
24,900	.0028	.0028	—.01	—.02	0	0
49,400	.0124	.0045	—.01	—.02	0	0
100,100	.0503	.0046	+.01	—.01	—.01	0
149,500	.0613	.0052	+.01	—.01	—.01	0
200,200	.0767	.0056	+.01	—.01	—.01	0
249,600	.0897	.0058	+.01	—.01	—.01	0
300,300	.1040	.0059	+.01	—.01	—.01	0
349,700	.1163	.0060	+.01	—.01	—.01	0
400,400	.1303	.0063	+.01	—.01	—.01	0
449,800	.1438	.0065	+.01	—.01	—.01	0
500,500	.1585	.0069	+.015	—.01	—.01	0
525,200	.1642	.0077	+.02	—.01	—.01	0
549,900	.1713	.0080	+.02	—.01	—.02	0
574,600	.1812	.0096	+.02	—.01	—.02	0
600,600	.1888	.0128	+.02	—.015	—.02	0
625,300	.1963	.0163	+.02	—.02	—.02	0
650,000	.2038	.0205	+.02	—.02	—.02	0
674,700	.2110	.0258	+.02	—.02	—.02	0
699,400	.2194	.0304	+.02	—.02	—.02	0
725,400	.2293	.0381	+.02	—.0175	—.02	0
750,100	.2370	.0429	+.02	—.02	—.02	0
774,800	.2466	.0517	+.02	—.015	—.02	0
799,500	.2573	.0595	+.025	—.0175	—.02	+.01
850,200	.2768	.0710	+.025	—.02	—.02	+.01
899,600	.2963	.0850	+.025	—.0225	—.02	+.02
950,300	.3173	.0940	+.025	—.0225	—.02	+.04
999,700	.3460	.1106	+.02	—.0225	—.02	+.04
1,050,400	.3988	.1463	+.02	—.0275	+.38	+.03
1,092,000	.4538	.1713	+.02	—.11	+.78	.0

Failed by crippling of top flange near pin-plates at one end.

Elastic limit 18,700 lbs. per sq. in. Maximum strength 33,950 lbs. per sq. in. The compression and deflections, plotted on load, are shown in Fig. 19a. A view of the crippled end of the chord is shown in Fig. 19b.

The crippling loads per square inch of section found in the above-recorded 19 tests are plotted in the diagram, Fig. 20, on the respective values of l/r as abscissas. Wrought iron and steel, respectively, are indicated by round and square dots. The tests which represent crippling of the body of the test-member are specially marked by a surrounding circle.

Table II. has been prepared to show in a general way how the results obtained compare with the crippling strength indicated by the formulas introduced by Mr. T. H. Johnson* and the late Prof. J. B. Johnson.†

The straight-line formula recommended by Mr. T. H. Johnson is, for hinged ends,

$$p = K - 0.4932\sqrt{\frac{K^3}{27\,E}}\,\frac{l}{r}$$

*In his paper "On the Strength of Columns: Discussing the Experiments which have been accumulated, and Proposing New Formulas," read June, 1885, published in Trans. Am. Soc. C. E. July-Dec., 1886, Vol. XV., pp. 517-531.

†In "Theory and Practice of Framed Structures."

TABLE I.—SYNOPSIS OF TESTS OF BUILT COMPRESSION MEMBERS.

	Division.	Bridge No.	Member.	Material.	Length, inches.	Area, sq. ins.	Section.	r	$\frac{l}{r}$	Elastic limit, lbs. per sq. in.	Yield point lbs. per sq. in.	Maximum lbs. per sq. in.	Remarks.
1.	C. & M. V.....	44	Post	Iron	300	17.78	I	3.05	98	....	18,000	24,800	Buckled as a w without marke local crip-pling.
2.	C., St. L. & P..	50	Post	Iron	300	17.56	„	2.49	120	....	20,500	26,350	
3.	O. C.	..	Post	Steel	156	15.25	„	1.87	83	....	23,600	34,270	
4.	O. C.	..	Post	Steel	156	14.15	„	1.87	83	....	24,700	32,900	
5.	Cincinnati	75	Chord	Iron	237½	25.4	∏	5.55	43	13,800	14,750	23,290	Failed in end detail.
6.	C., R. & C.....	25	Chord	Iron	175⅝	22.8	„	4.88	36	13,200	15,300	28,220	Failed in end detail.
7.	T., W. V. & O...	1	Chord	Iron	223⅜	27.95	„	5.69	39	15,200	19,700	29,210	Failed in end detail.
8.	T., W. V. & O...	3	Chord	Iron	223⅜	27.95	„	5.69	39	15,200	17,900	29,900	Failed in end detail.
9.	T., W. V. & O...	17	Post	Steel	300	14.22	⊢⊣	3.10	97	12,600	17,550	27,790	Outstanding legs of a gles buckled near cen.
10.	T., W. V. & O...	18	Chord	Iron	289¼	28.38	∏	6.34	46	16,700	18,500	26,200	Failed in end detail.
11.	T., W. V. & O...	18	Chord	Iron	289¼	28.20	„	6.33	46	15,950	19,500	26,230	Failed in end detail.
12.	T., W. V. & O...	21	Chord	Iron	163¹⁵/₁₆	27.46	„	5.65	29	17,300	20,000	30,770	Failed in end detail.
13.	T., W. V. & O...	24	Chord	Iron	242⅝	27.28	„	5.80	42	20,150	22,000	28,200	Lower flanges buckled near center & end detail showed signs of failure.
14.	C. & M. V.....	31	Chord	Iron	190¼	21.78	∏	4.71	40	17,200	20,650	26,260	Lower flanges buckled near center.
15.	C. & M. V.....	104	Chord	Iron	249	30.81	∏	6.51	37	17,000	24,300	30,300	Lower flanges buckled near end.
16.	New. & Cin.....	..	Chord	Steel	297½	29.36	„	6.44	46	25,500	28,900	30,640	Lower flanges buckled near center.
17.	New. & Cin.....	..	Post	Steel	168	15.80	][	3.72	45	19,000	22,100	31,680	Crippled near center.
18.	Louisville	20	Chord	Steel	249	32.11	∏	7.29	34	22,600	26,500	31,980	Lower flanges buckled near center.
19.	Louisville	20	Chord	Steel	249	32.16	„	7.20	35	18,700	23,300	33,950	Top flange buckled near end.

TABLE II.—COMPARISON OF RESULTS OF TESTS OF FULL-SIZE COLUMNS WITH FORMULAS OF T. H. JOHNSON AND J. B. JOHNSON.

Formula No. 1 [T. H. Johnson]

For Iron: $p = 37{,}000 - 128\,\frac{l}{r}$

For Steel: $p = 47{,}000 - 178\,\frac{l}{r}$

Formula No. 2 [J. B. Johnson]

For Iron: $p = 30{,}700 - 0.51\left(\frac{l}{r}\right)^2$

For Steel: $p = 39{,}900 - 0.82\left(\frac{l}{r}\right)^2$

Test.	Material.	Area of section sq. in.	$\frac{l}{r}$	Crippling load by formula. No. 1.	Crippling load by formula. No. 2.	Elastic limit. Lbs. per sq. in.	Elastic limit. Per cent. of crippling. Form No. 1.	Elastic limit. Per cent. of crippling. Form No. 2.	Elastic limit. Per cent. of crippling. Actual.	Yield point. Lbs. per sq. in.	Yield point. Per cent. of crippling. Form No. 1.	Yield point. Per cent. of crippling. Form No. 2.	Yield point. Per cent. of crippling. Actual.	Maximum. Lbs. per sq. in.	Maximum. P. ct. of calcul'd. Form No. 1.	Maximum. P. ct. of calcul'd. Form No. 2.
1.....	I	17.78	98	24,500	25,800					18,000	73	70	73	24,800	101	96
2.....	I	17.56	120	21,600	23,400					20,500	95	88	78	26,350	122	113
3.....	S	15.25	83	32,200	34,300					23,600	73	69	69	34,270	106	100
4.....	S	14.15	83	32,200	34,300					24,700	78	72	75	32,900	102	96
5.....	I	25.4	43	31,500	29,800	13,800	44	46	59	14,750	47	49	63	23,290	74	78
6.....	I	22.8	36	32,400	30,000	13,200	41	44	47	15,300	47	51	54	28,220	87	94
7.....	I	27.95	39	32,000	29,900	15,200	48	51	52	19,700	62	66	67	29,210	91	98
8.....	I	27.95	39	32,000	29,900	15,200	48	51	51	17,900	56	60	60	29,900	93	100
9.....	S	14.22	97	29,700	32,200	12,600	42	39	45	17,550	59	55	63	27,790	94	86
10.....	I	28.38	46	31,100	29,600	16,700	54	56	64	18,500	59	63	71	26,200	84	89
11.....	I	28.20	46	31,100	29,600	15,950	51	54	61	19,500	63	66	74	26,230	84	89
12.....	I	27.46	29	33,300	30,300	17,300	52	57	56	20,000	60	66	65	30,770	92	102
13.....	I	27.28	42	31,600	29,800	20,150	64	68	71	22,000	70	74	78	28,200	89	95
14.....	I	21.78	40	31,900	29,900	17,200	54	58	65	20,650	65	69	79	26,260	82	88
15.....	I	30.81	37	32,300	30,000	17,000	53	57	56	24,300	75	81	80	30,300	94	101
16.....	S	29.36	46	38,800	38,200	25,500	66	67	83	28,900	74	76	94	30,640	79	80
17.....	S	15.80	45	39,000	38,200	19,000	49	50	60	22,100	57	58	70	31,680	81	83
18.....	S	32.11	34	40,900	38,900	22,600	55	58	71	26,500	65	68	83	31,980	78	82
19.....	S	32.16	35	40,800	38,900	18,700	46	48	55	23,300	57	60	69	33,950	83	87

[illegible] states that the K assumed in his study is practically the modulus of transverse rupture of the material. No tests of the material used in the members were made that would indicate what that value was, but Mr. James Christie made some tests of similar material (see Trans. Am. Soc. C. E., August, 1884, Tables 3 and 4, "The Strength and Elasticity of Structural Steel and its Efficiency in the form of Beams and Struts"), in which 37,000 lbs. was the average of 27 iron specimens and 47,000 lbs. the average of 29 steel specimens. The average E he found to be 28,000,000 for iron and 30,000,000 for steel. The value of E was not determined in specimen tests of iron in the members here represented, but that of the steel used was found to average 29,600,000. The measurements obtained in the tests seem to indicate that the value of 28,000,000 is a fair one for the iron used. The numerical values of Formula No. 1 of Table II. are therefore based on values of 37,000 for K and 28,000,000 for E for the iron columns and 47,000 and 29,500,000 for K and E respectively for the steel.

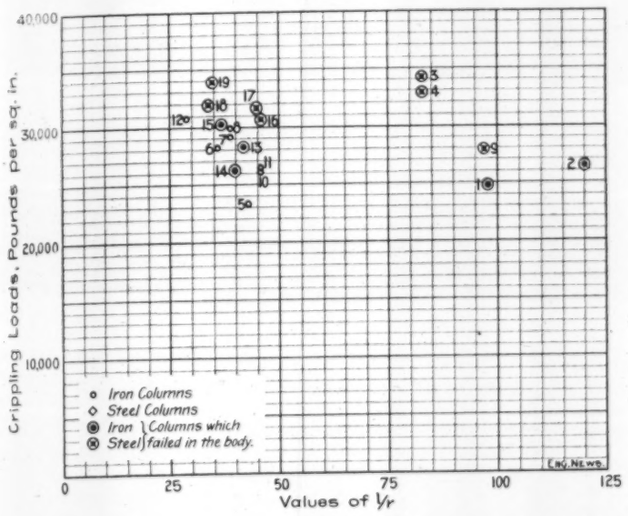


FIG. 20. CRIPPLING LOADS OF 19 PIN-END BUILT COLUMNS TESTED BY BRIDGE DEPARTMENT OF PENNSYLVANIA LINES WEST OF PITTSBURG, 1888-1900.

Maximum load in pounds per square inch plotted on length-ratio l/r.
Columns 14.22 to 32.16 sq. ins. in cross-section, 13 ft. 7 5-16 ins. to 25 ft. 0 ins. long c. to c. of pins.
Material, wrought-iron and steel.

In "Theory and Practice of Modern Framed Structures" Prof. Johnson says the term f in his formula

$$p = f - \frac{f^2}{64\,E}\left(\frac{l}{r}\right)^2$$

should be taken as the elastic limit of the material in tension. An average of 68 tests of the iron used in the members tested gives 30,700 lbs., while an average of 221 tests of the steel used indicates an elastic limit of 39,900 lbs., and these figures have been used with the same values of E as in the other case for finding the numerical value of Formula No. 2.

For chords 18 and 19, material was ordered 20 ins. longer than required and tension tests made of specimens cut from the actual material in the finished chords. The other tests used in determining the averages given above were of specimens from material rolled for the same and similar sections of the respective bridges represented.

STREET CARS of the "pay-as-you-enter" type are now in use on the Cottage Grove Ave. line in Chicago, and have proved satisfactory in the first few weeks of service. They appear to be filled and emptied as easily and rapidly as the ordinary cars of the same size, and it is expected that there will be greater promptness when the public becomes fully educated as to their peculiar features. The cars are 45 ft. 9 ins. long over all, 32 ft. 6 ins. long over the body, 9 ft. wide over all, and 8 ft. 2½ ins. wide inside. They have transverse seats on each side of the central gangway, and a longitudinal seat for four passengers at each corner. The total seating capacity is for 44 passengers. About 150 cars are in service, and 150 more are being built for the Indiana Ave. line, both lines running on Wabash Ave. in the downtown district. The cars of this type for the Madison Ave. line in New York were described and illustrated in Engineering News of Nov. 21.

NATURAL CONDITIONS AFFECTING ENGINEERING WORK IN ALASKA.

By CAPT. GEO. B. PILLSBURY,* Assoc. M. Am. Soc. C. E.

In view of the number of engineering enterprises already under construction or projected in the interior of Alaska, and the probability of many others in the future, the writer has thought that a brief account of the conditions to be expected in that country may be interesting and useful.

It should first be observed that the territory extends through more than a thousand miles of latitude, and that the variation in climate from the northernmost to the southernmost portions is comparable to the change from North Dakota to Texas. In the same manner that conditions in the Gulf States and in the Middle West may be broadly outlined with sufficient definiteness, the conditions on the Pacific coast of Alaska and in the interior may be stated in general terms.

PACIFIC COAST.—On the Pacific coast of Alaska, which must be the starting point of any trunk line of railroad, the climate and labor conditions are generally similar to those in Oregon and Washington. Practically all points can be reached in from four to seven days by steamer from Seattle. The freight rates are reasonable. The scarcity of labor now existing in the Pacific States extends naturally to this section in a greater degree. Wages for ordinary labor run from $3 to $4 per day, the men paying from $6 to $7 per week for board. The climate is mild, as far as concerns temperature, but the precipitation is very high. The rains are, however, more continuous than heavy. The snowfall is in most places considerable, and snow stays late in the spring. Work can best be executed to advantage between the first of May and the first of December, making a working season of about seven months. The rugged Coast Range of mountains lines the Pacific coast of Alaska practically without a break, making construction in this section costly. In general, the engineer will do well to add 50 per cent. to the unit costs of corresponding work in the United States to obtain the cost of work in this locality.

INTERIOR.—While conditions on the Pacific coast of Alaska may be described as but comparatively little out of the normal, the same cannot be said of the vast country in the interior. In this section the engineer is confronted with a short working season, with slow and costly transportation and communication, with excessive cost of labor, in many places with permanently frozen ground and, in all bridge work, with very heavy ice. As a partial offset to these disadvantages, the climate is exceptionally healthful, and the labor, although costly, is efficient. The men will willingly undergo necessary hardships and privation, and, on pioneer work, are extremely resourceful.

The working season for all outside work does not extend beyond a period from the first of June to the first of October. The cost of transportation varies, of course, with the locality. To coast ports of Bering Sea, the rate of freight, including lighterage, is from $10 to $15 per ton. To interior Yukon and Tanana River points, the rate is from $60 to $150 per ton, depending upon the classification, route and destination. From the river ports to points inland, wagon transportation, on good roads, costs at least $2 per ton mile, and one may consider himself extremely fortunate to secure such a rate. Wages, for ordinary labor, run from $4 to $5 per day, board being furnished, at a cost of from $1.50 to $3 per day.

The peculiarly interesting features in regard to ordinary construction work are the frozen soil and heavy winter ice. It can by no means be said that all the ground is permanently frozen, but, it may be stated that, as a rule, all north hill sides, and all valley bottoms, are permanently frozen to an unknown depth. Shafts sunk 300 ft. have, in some cases, not penetrated the frozen ground. Masses of ice, in some cases blue and clear, are occasionally met with in the frozen soil, these being evidently ponds that were subsequently covered with alluvial deposits before the ice contained therein had melted. The ground of this character is covered with a thick carpet of moss and vegetation, that effectually prevents it from thawing during the summer. Even in August, on stripping the moss, the ground is ordinarily found to be glistening with frost at the surface. The prevalence of the frozen ground, with consequent lack of drainage, gives rise to the swampy characteristics of the interior country.

As is well known, frozen earth is a difficult material to handle. It cannot be blasted to advantage, as it has no lines of fracture, but is homogeneous and somewhat elastic. Excavation in frozen ground is necessarily expensive. If the excavation is of no great depth, it is preferable to strip the ground early in the year, and let it thaw in the sun. Through lack of drainage, the ground, if not gravel, will be extremely soft mud when thawed, and horse-scrapers will work in it only at great disadvantage, if indeed they can work in it at all. In placer mining, in this region, steam cable scrapers are frequently used.

In deep excavation, the ground must practically be thawed with steam points. A half-inch steam pipe, ordinarily of special construction, is driven into the ground to be excavated and steam is forced in. About 1.5 rated HP. is required to each point, and the amount thawed varies from 1.5 to 4 cu. yds. in 12 hrs. At the current prices for fuel and labor, the cost of thawing by steam, including the plant, is in the vicinity of 50 cts. per cu. yd. The formation of ice in the streams has peculiar characteristics, due to the long-continued cold weather. All but the largest streams to freeze to the bottom, in places at least. As the stream is thus dammed, the ground water in and along the bed overflows the ice, frequently building it up to many times the summer depth of the stream. In the spring the ice melts out without running and jamming. On the larger streams, however, such as the Yukon and the Tanana, the ice runs and jams in the spring to a very noteworthy extent. A common effect of the ice in ordinary streams, and even in slack water, is to heave up any piling that may be driven. The effect is easily understood, and requires no explanation, but it renders pile trestles in water quite undesirable. They can be saved, if there is no current, or movement of the ice, by cutting the ice around the piles in the spring, but the expense of this maintenance is considerable.

As might be expected, the timber in the interior is poor and scanty. On the coast of Bering Sea there is practically none. In the great river valleys spruce may be had, but it is exceptional to find sticks that will square over 10 × 10 ins. for a length of 20 ft. Timber is very little subject to decay.

For construction in the interior, the unit costs for similar work in the United States should be multiplied by three or four, to obtain a reasonably safe rough estimate of cost.

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THE WATER-POWER DEVELOPMENT OF THE GREAT NORTHERN POWER CO., NEAR DULUTH.

A water-power development of exceptional interest on the St. Louis River, near Duluth, Minn., is now under way. While the plant, as now built and equipped, has a capacity of 30,000 HP. (with provision for a future capacity of about 80,000 HP.) the entire project calls for the ultimate development of about 200,000 HP. It also includes the transmission of electric current to Duluth (14 miles distant) for general power purposes, including the operation of water-works pumping engines. Its transmission to the Mesaba, Vermillion and other iron mining regions is contemplated, as it is suggested that it might be used to operate hoisting plants and excavating machinery, as well as for general lighting and power purposes. The current may also eventually be transmitted to St. Paul and Minneapolis (120 miles). The total amount of power would be generated at three different plants (two on the St. Louis River), under heads of 740 ft., 378 ft., and 70 ft.; these plants would have a generating capacity of 110,000 HP., 80,000 HP. and 12,500 HP., respectively. The second of these plants is now practically completed, and has its first instalment of machinery ready for operation, with a capacity of 30,000 HP. The development of the water-power is being done by the Great Northern Power Company, of Duluth, Minn., while the utilization of the power (including the establishment of industrial enterprises), is in the hands of the Great Northern Development Co. Mr. C. C. Cokefair is President and General Manager of both companies. For information we are indebted to Mr. F. A. Cokefair, Assoc. M. Am. Soc. C. E., Chief Engineer of the Great Northern Power Co.; and to Mr. Thomas Pettigrew, M. Am. Soc. C. E., Chief Engineer of the National Railway Construction Co., of Boston and Duluth, which company had the general contract for the construction work. The St. Louis River has a watershed of about 3,700 sq. miles, with a mean annual rainfall of 30 ins., and an average run-off of 12 to 18 ins. The watershed comprises numerous small lakes and large swamp areas. The river drains a plateau at an elevation of 800 to 1,000 ft. above Lake Superior, and extending to within a short distance from the lake. From this plateau there is a steep fall through a deep gorge extending to the lake, while on the river itself the fall can be utilized for power development at two levels. Storage reservoirs will be required to equalize the flow and to render available a large part of the run-off, but the drainage area is said to offer exceptionally favorable opportunities for establishing such reservoirs at low cost. The first of these would be at Rice Lake, 10 miles north of Duluth, but this will not be required for the first installation of 30,000 HP.

The plans for the largest of the three plants above mentioned, provides for taking water from the river at a point above Thomson and leading it by a canal to the edge of the bluff overlooking the city of Duluth, where it will be carried by penstocks to a power plant on the lake shore, with a direct head of about 740 ft. This would take the water from a drainage area of about 1,200 sq. miles of the total area.

The plant which is now completed is located near Thomson, Minn., and its general features are shown by the map and profile in Fig. 1. It is entirely independent of the large project noted above, and will take the water from 2,375 miles of drainage area below the area tributary to the latter.

The third projected plant, which is also shown in Fig. 1, will consist of a dam farther down the river than the present plant, and just above Fond du Lac, Wis. This would give a head of 70 ft., which would be utilized by a power house at or just below the dam, where pulp or paper mills may be established.

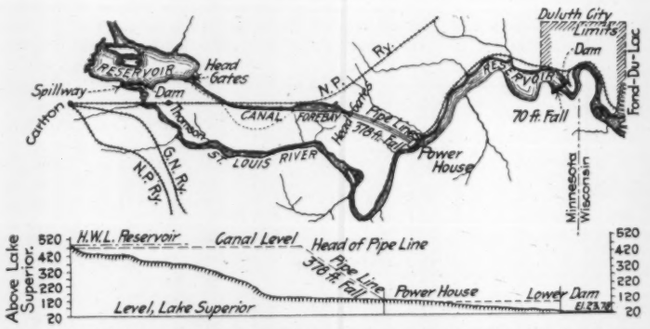


FIG. 1. PLAN AND PROFILE OF WATER POWER DEVELOPMENT NEAR DULUTH, MINN.; GREAT NORTHERN POWER CO.
F. A. Cokefair, Assoc. M. Am. Soc. C. E., Chief Engineer.

With this review of the complete project, we may turn now to the existing plant, Fig. 1. At the outlet from a small lake near Thomson, a dam has been built to form a large lake or service reservoir. From the head gates at this reservoir a canal 2½ miles long has been built, terminating in a forebay at the lower headgates. Beyond this, pipe lines or penstocks are laid to the power house in the bottom of the gorge, the available fall from the forebay to the power house being 375 ft. The service reservoir (¾-sq. mile in area) serves to meet the variable demands for power during 24 hours, without the loss of water over the spillway of the dam. It has sufficient capacity to operate the ultimate equipment of 80,000 electrical HP. continuously for 24 hours without supply from the river, and will also operate it for a sufficient time to receive a supply from the storage reservoirs in case of emergency without interruption to the service.

The main dam across the river is built of concrete, on solid rock foundations. It is 1,120 ft. long, with a spillway 365 ft. long, 38 ft. in height, and 42 ft. wide at the base, with a curved crest at an elevation of 484 ft. above Lake Superior. It is shown in Fig 2. In this dam are three sluice gates, each 7 × 9 ft. (at the right in Fig. 2). The eastern retaining wall to the main dam is 609 ft. long, and the western wall 14[illegible] ft. long, making the entire length of the structure across the river 1,120 ft. The retaining wall to the dam and all fills about the reservoir [illegible]e 6 ft. higher than the crest of the spillway. The concrete is composed of Universal Portland cement, sand and washed Lake Superior gravel in proportions of 1:2:4. A rock ridge extending [illegible]st from the main dam has been cut down to an elevation of 485 ft. above the lake (or 1 ft. higher than the spillway crest of the main dam) for a distance of 1,000 ft., so as to provide additional overflow capacity in case of excessive floods. Fig. 2 is a view of the spillway section and gate openings. Two of the small dams are of earth construction, faced with rip-rap; these were built under contract by Truax & Thomas, of Duluth. One was a rock-fill dam, with a central core wall, and the other seven dams are of concrete. The total amount of concrete in all the dams is about 25,000 cu. yds.

Ice conditions on the river have been carefully observed, and provision made to prevent interruption of the service from this cause, but neither surface nor needle ice are expected to be especially troublesome. Water is drawn from the bottom of the reservoir into the canal through submerged arches at the head gates. The design is such as to keep velocities of approach low, so that very little ice or other floating material is expected to get into the canal. The water level of the canal is 472 ft. above Lake Superior, thus allowing a draft of 12 ft. on the service reservoir below the crest of the dam. There are eight gate openings, three of which are fitted with gates 6 × 9 ft., while the others are temporarily closed.

The canal is in earth for about a mile from the head gates, and this section is 36 ft. wide on the bottom, with a depth of 15 ft. and side slopes of 1 on 2. The maximum velocity through this section, when the canal is running full, will be about 2.75 ft. per second, and the discharge 2,000 cu. ft. per second; the slope will be 0.00015%; coefficient of roughness, 0.025. The remainder of the canal (about 3,500 ft.) is excavated through slate rock, and the section is rectangular, 30 ft. wide and 15 ft. deep. The surface stratum of earth is sloped 1 on 2 to a berm along the edge of the canal. The maximum velocity in the rock section, when running full, will be about 7.50 ft. per second, and with the same discharge and slope as above, but with a coefficient of roughness of 0.035. Near the end of the canal is a concrete spillway to prevent flooding or overflow at the forebay.

The forebay at the east end of the canal has an area of 40 acres, and is formed by an earth embankment 3,500 ft. long and 30 ft. in average height, having a core wall of planking. About 85,000 cu. yds. of quicksand were removed from under the location of this embankment to secure a good foundation. The embankment is 25 ft. wide on top, with a water slope of 1 on 2½, and an outside slope of 1 on 1½. Both slopes are covered with rip-rap. This forebay will provide for sudden fluctuations of station load without changing the head gates at the canal, and water will be drawn from the forebay into the pipe lines through head gates with concrete curtain walls. The bottom of the pipes as they enter the head gates will be at an elevation of 450 ft. The water in the forebay can be drawn down from the water level of 472 ft. to an elevation of 460 ft., affording a storage capacity sufficient to operate the station for several hours.

The lower head works provide openings for eight pipe lines ultimately, but only three gates and pipes have been built. The pipe lines are each 7 ft. internal diameter and one mile long, laid in trenches and buried. Wooden pipe, of California redwood staves 3½ ins. thick, is used for

FIG. 2. DAM AT THE ST. LOUIS RIVER AT THOMSON, MINN.
(At the right are the sluice gates.)

4,000 ft. of the distance, and the pressures upon the lower end of the wooden pipes will be equivalent to about 150 ft. head. The lower 1,000 ft. of each pipe line is of riveted steel plates, and operates under heads of from 150 ft. to 375 ft.

One of the special and most interesting features of the plant is the pressure equalizing tank connected to the penstocks. About 500 ft. from the power house, and at an elevation of 260 ft. each pipe is connected by a header and valve with a cross receiver, which in turn is connected to a 6-ft. standpipe 235 ft. in height from the bottom of its foundations to the top of a steel tank 30 ft. diameter and 70 ft. high. The function of this standpipe is not only to relieve excess pressures due to sudden drop of load in the station, or other causes, but also to maintain pressures and facilitate speed regulation in the station when loads are suddenly thrown on. The 30 ft. tank has sufficient capacity in a height of 46 ft. from the bottom (between elevations 426 and 472 above Lake Superior level) to supply water to the wheels to meet a sudden demand for power up to 10,000 HP. It will also maintain the head required by the water wheel governors for good regulation, while the water in the penstocks is accelerating to the speed required to meet the added demand. The water in the standpipe and tank will be kept from freezing by allowing warm air to rise in the air space between the pipe and the lagging, the heat being supplied by steam from the heating plant in the power station, 500 ft. distant. The equalizing tank and its connections are shown in Fig. 7.

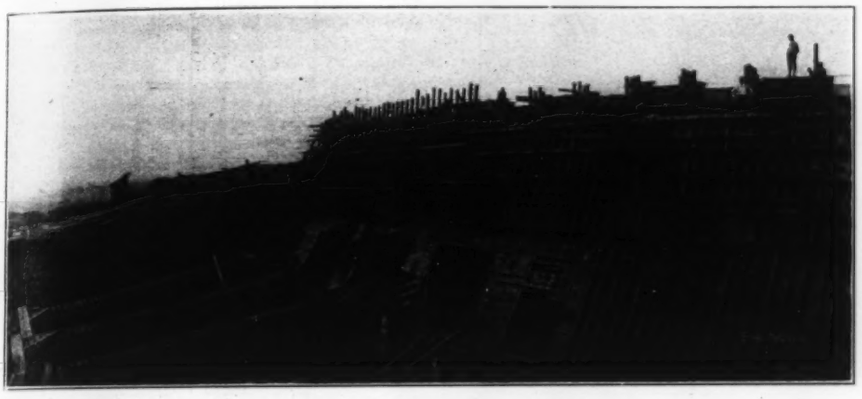


FIG. 3. LOWER HEAD GATES AND WOODEN STAVE PIPE LINES.

CONSTRUCTION METHODS.

The general contract was awarded to the National Railway Construction Co., of Boston and Duluth, and the work was under the direction of its Chief Engineer, Mr. Thomas Pettigrew, M. Am. Soc. C. E. On the site of the large dam, before construction began, there was a timber dam about 12 ft. in height, making a small reservoir for the use of the St. Louis River Slate Brick Works. Under arrangement with this company a section of the timber dam was blown up in January, 1905; this was the first step towards construction. A cableway from the Flory Mfg. Co., 1,000 ft. span, with a capacity for 4-ton loads, was built so as to span the greater part of the large dam, nearly coincident with its longitudinal axis. The concrete mixing plant consisted of a cubical mixer of 24 cu. ft. capacity, and was so situated under this cableway that the skips could be handled by the cableway directly from the mixer. This is shown in Figs. 2 and 4.

There was a conveying system for the handling of all materials, and this (as well as the mixing plant) was operated by a single high-speed engine of about 20 HP. A canvas belt connected the mixing platform with the cement house, and the cement bags were placed on this belt by hand and dumped on the platform near the hopper. On the opposite side of the mixer were the stock piles of sand and washed Lake Superior gravel, and in a tunnel under these was operated a conveyor belt on which sand and gravel as needed were dropped from vertical chutes which were regulated very closely by hand devices. This conveyor belt carried the material horizontally to a vertical bucket conveyor, which raised it to bins of about 20 yds. capacity above the mixer. From the bins the material was dropped through chutes into hoppers, measuring exactly the quantity in each batch. Water was taken from an overhead tank through a simple automatic measuring device. The sand and gravel were loaded on bottom-dumping Rodger ballast cars from scows in Duluth harbor, and the cars dumped their contents from an overhead track onto stock piles.

The foundations of all dams were solid rock, lying very irregularly and giving very excellent security against sliding and bottom leakage. The only difficulty experienced in foundations was in the main part of the river, about 500 ft. across. About half of this was enclosed with an outer cofferdam of rock-filled crib and an inner dam of sheet piling and clay. There was much underflow through the gravel and boulders overlying the rock, and it kept two 8-in. centrifugal pumps busy to keep the water down, even when the dam was as tight as it could be made. This part of the dam was built in the spring and early summer of 1905. It included a set of three sluice gates, designed to carry the low water flow of the river while constructing the second half.

As 1905 was an exceptionally wet year, there was no low water during the summer, and it was not possible to send the river through the sluice gates provided for it. For that reason all the fall months were lost on the construction of the river section, and it was not possible to close the remainder of the cofferdam and send the river through the sluice gates until some time in December. Fig. 4 shows the condition of the work during high water at the dam in June, 1905. This half of the river was much harder to control than the first. No puddle material which could be procured at the site seemed capable of resisting the dissolving effect of the water flowing on the bed rock, and as the work was being done in the rigors of a northwest winter it was very trying. Finally, by excavating (under water) trenches 2 ft. wide to impervious material, and filling them with concrete set in the water, it was possible to pump out the work. Work was pushed day and night, both on foundations and setting concrete, with the thermometer often at 10° below zero, or even lower. Proper arrangements were made for heating the materials, and the forms were lined with tarred paper.

The dam was completed about March 10, 1907, and the plant removed later. Rising water compelled the closing of sluice gates about April 5, and the reservoir was allowed to fill. In all cases the concrete set in the coldest weather is said to appear fully as good as that set in the summer previous. Preparations were made in the late fall for protecting the winter work against freezing. Coils of steam pipe were placed in the bins above the mixer for sand and gravel, and also in the tunnel under the stock piles and in the water tank. Live steam was also introduced into the stock piles at any desired point by means of flexible hose and pipe. In this way it was possible to give the sand and gravel a temperature of about 50° and the water more than 100° F. This gave to the completed concrete a temperature sufficiently high to insure its setting in the paper-lined forms before frost could affect it. When work was stopped in any form, it was at once covered with canvas about 4 ft. above the surface of concrete, and salamanders with red hot coke were set on the concrete under the canvas. It was only necessary to keep this up for about ten hours, as the setting was beyond the power of frost by that time. The same general method was followed with all the concrete work done in the winter and was found to be eminently successful. In no case did any poor work result on account of severe weather.

The smaller concrete dams were, for the most part, built from a portable mixing plant of ½ cu. yd. capacity, and a derrick was used to set the concrete in place. Several dams were built

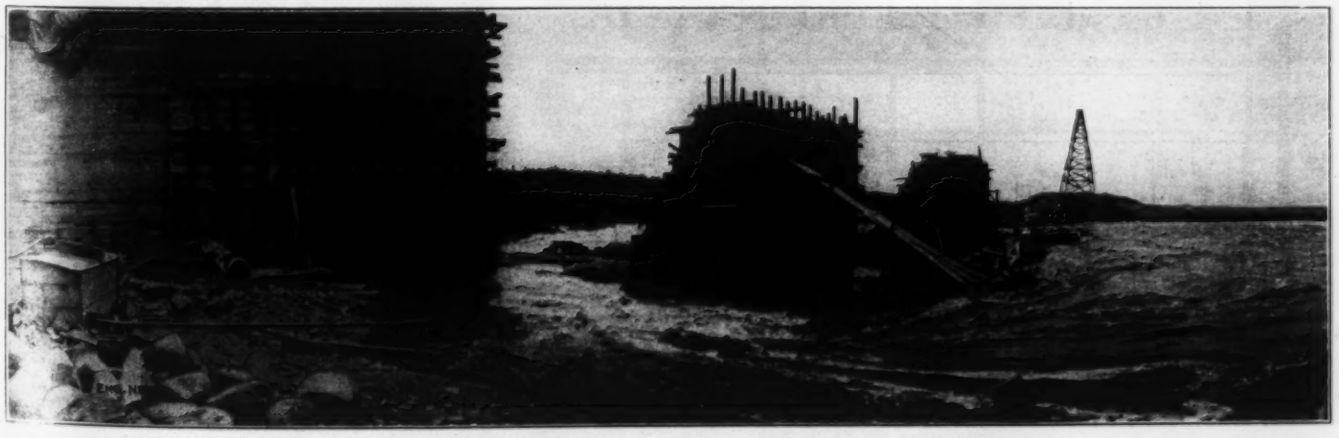


FIG. 4. HIGH WATER DURING THE CONSTRUCTION OF THE ST. LOUIS RIVER DAM.

from the same setting of the concrete plant by laying a car track and carrying the mixed concrete in skips on flat cars to the point where used. As the cement took about three hours to get its initial set, it could be moved some distance without risk of injury.

The work at the upper head gates was all done by the construction company during February and March, 1906. The work was mostly reinforced concrete, and was built with a ½-yd mixer plant. Material was hauled by wagons, and small cars operated by a friction hoist raised it to the mixer platform. On account of cold weather everything was housed in, and the same precautions were taken for heating materials as at dam. The controlling gates were built by the Coffin Valve Co., and set by the construction company. The work was completed and it was subjected to a full head of water when the reservoir was filled in April, 1906.

The canal and forebay were originally let by contract to Porter Brothers, of Duluth, but were finished by the National Railway Construction Co., which took them over by arrangement with the contractors on Oct. 15, 1905. Work was pushed night and day from that time until November 1, 1906, at which time water was let through. This was a very difficult piece of work, the material being slate rock on edge overlaid by mud of various depths according to height of ridges. No one particular method was followed. Steam shovels were used where practicable, and much material was handled with ordinary derricks and gangs of men. Some use was also made of orange-peel buckets. Drilling was done by compressed air from a central plant. The difficulties of this canal construction were underestimated at the start, and the plant was not as well suited to the work as might have been. Such a large percentage of the earth material was of a soft quicksand nature that it was not thought advisable to use it in the forebay bank. This bank was, therefore, built of gravel and clay taken from a borrow pit by ordinary steam shovel work, with small locomotives and cars. The water side was sloped 1 on 2½, and was heavily faced with rock from canal. About 80,000 cu. yds. of soft material was excavated on the side of the forebay bank for fear of possible settlement, and a core wall of timber was built longitudinally through the center of bank. The lower head gates were built by the construction company with its own men. The work was principally reinforced concrete and was done during the summer of 1906.

Three lines of 7-ft. redwood stave pipe, each about 4,000 ft. long, were let to the Excelsior Wooden Pipe Co., and completed by it during 1905 and 1906. The steel penstocks were let to the Riter-Conley Mfg. Co., and completed in October, 1906. Fig. 3 shows the lower head gates and wooden stave pipe lines. The substation in Duluth was built by J. W. Hilliard, of Duluth. The power house and all transmission and distribution lines were built by the National Railway Construction Co.

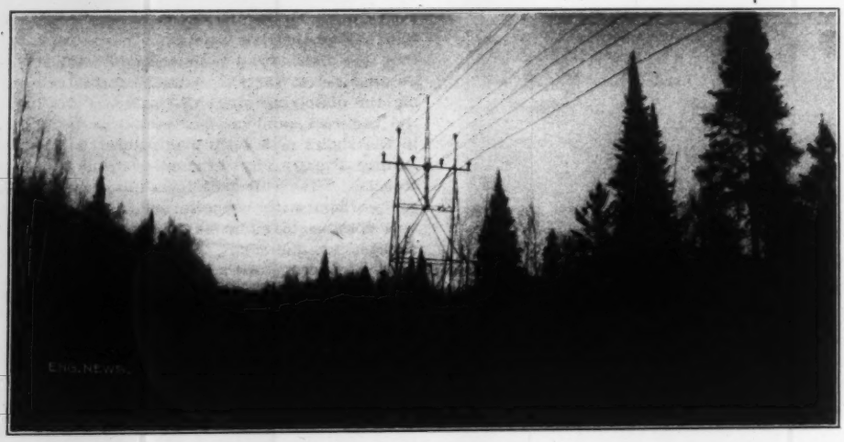


FIG. 5. TRANSMISSION LINE TO DULUTH.

POWER PLANT.

The power house has concrete foundations and a brick and steel superstructure. It is now built for four 10,000 HP. units and two exciters, but the plans provide for an ultimate extension up to eight units. Sections of the present installation are shown in Fig. 6. Each of the penstocks after entering the power house is reduced for connection with a 72-in. hydraulic valve. Each penstock below the valve connects with a 13,000 HP. water-wheel or turbine, with vertical shaft, designed for a speed of 375 revolutions per minute. The shaft is 23 ft. in length, and its upper end carries the internal revolving field of the generator, located on the main power house floor above. The center of each water-wheel is 11 ft. above the low water level (elevation 92) in the river, and the draft tubes discharge directly into the river. The turbines are of the Escher-Wyss type, and were built by the Allis-Chalmers Co., of Milwaukee, Wis. The speed is regulated by governors of the Escher-Wyss type. Attached to the housing of each wheel are two relief valves,

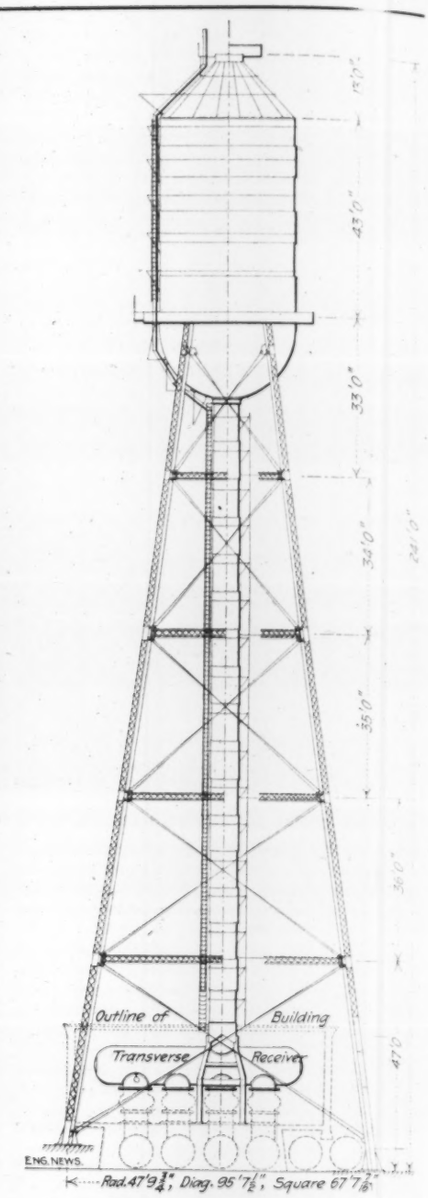


Fig. 7. 240-Ft. Standpipe and Elevated Tank Equalizing Pressure at Lower End of Pipe Lines. Chicago Bridge & Iron Works, Chicago, Builders.

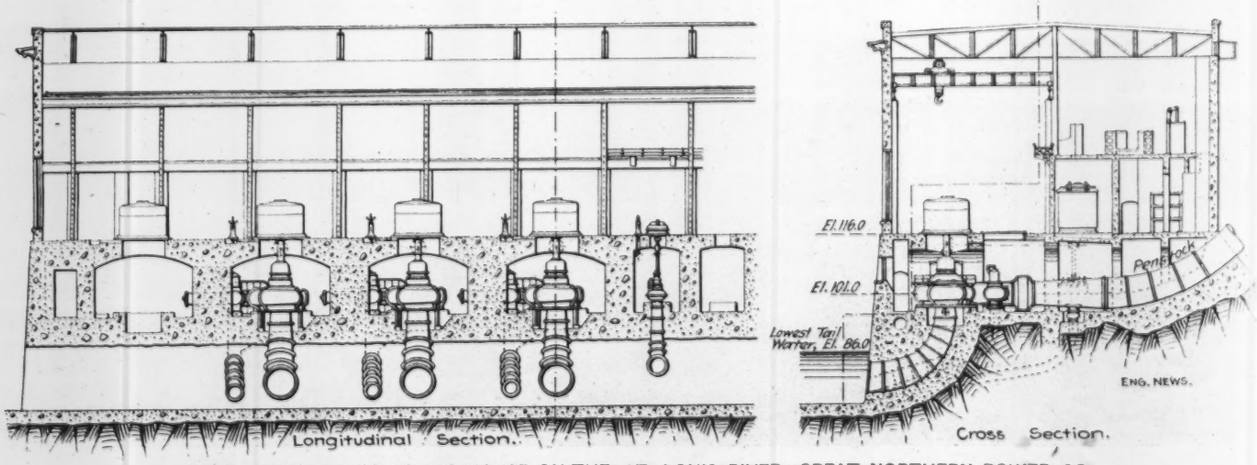


FIG. 6. SECTIONS OF POWER HOUSE ON THE ST. LOUIS RIVER; GREAT NORTHERN POWER CO.

which can be operated either by direct pressure or from the governor, as may be desirable. These valves will be set to discharge at a pressure equivalent to a head which will not overtop the standpipe. Pipe connections for the exciter wheels are made from the main penstocks. These wheels are similar to the main turbines, and each is designed to drive a 250 KW., 125-volt exciter at 500 revolutions. The main generators are of the General Electric Co.'s make, of 7,500 KW., wound for a three-phase current of 6,600 volts, 25 cycles. Water-cooled oil transformers are installed, with taps so that the voltage can be varied between 30,000 and 60,000 volts. All the high tension wiring, bus-bars, transformers and lightning arresters are in brick compartments, and the whole station is under central control from the switchboard.

The transmission line to Duluth, Fig. 5, is 14 miles long, with steel towers 40 to 60 ft. high, spaced about 300 ft. to 1,000 ft. apart. There are two circuits, each of which can take the full load in case one of these should break down. The voltage is 30,000, but the line is designed so that the voltage can be raised to 60,000. The Duluth substation has the same general compartment wiring scheme as the power station, with high and low tension bus-bars and switchboard for both alternating and direct current service. At the substation the voltage will be stepped down through transformers of the same design and capacity as used in the power house. The line is extended to Superior, crossing the harbor by 1,200 ft. of submarine cable.

TURBINES.

The turbines, of 13,000 HP. each, are said to be the most powerful turbines ever built with a single wheel. As already noted, the plans provide for the installation of eight of these large units eventually, with four smaller turbines to drive exciter units for the large generators. The turbines, together with their accessories (excepting the governors) are placed in the basement of the power house, leaving the main floor free for the electrical machinery. The hydraulic governors are on the main floor beside the generators. The pumps for providing oil under pressure for the governors and thrust bearings, and also the oil pressure tanks, etc., are in a separate basement.

The following particulars of these machines have been furnished by the builders:

The turbines are of the inward discharge, scroll case type, with a horizontal casing and vertical axis. In order to obtain the high efficiency demanded at part-gate opening, each turbine is provided with swivel admission gates at the scroll opening. The scroll case is of cast steel and is split to provide for easy dismantling without disturbing the electrical apparatus above. The spiral case rests upon a cylindrical base, which in turn rests upon a concrete foundation. The guide case is within the spiral case and contains the speed gates. It carries the entire weight of the rotary parts and transmits the load on the thrust bearing to the foundations. Inside the guide case are the movable guide vanes which form the speed gate, and each of these vanes, with the pivot on which it turns, is made in one piece of forged tool steel. The pivots, which have short bearings to keep them in proper alinement, project through stuffing boxes; they are all connected to a concentric gate ring which is operated by means of two main shafts and cranks. These crank shafts are connected to the operating levers and to the governor relay. This arrangement of the gate ring and its connections, by bringing them outside of the wheel case, has the advantage of leaving them free from the wear and tear due to the water and keeping them open to inspection at all times, even during operation.

The crown plate containing the stuffing box for the main shaft and manholes is placed above the guides. The yoke supporting the thrust bearing, which carries the weight of the rotary parts, rests on the guide case. The yoke encases the main steady bearing of the turbine shaft, and is split to facilitate erection and dismantling. The steady-bearing consists of a two-part box, lined with babbitt and lubricated by oil in circulation, an oil catcher guiding the used oil into a basin. The main shaft is constructed of steel in one piece. At its lower end is a flange to which the runner is bolted. This arrangement, with the splitting of the casing, yoke, and other parts, permits of the dismantling of the turbine without disturbing the rotor of the generator. The turbine shaft collar transmits the weight of all the rotary parts to the thrust bearing. The thrust plates are of close-grained cast-iron, with chambers and grooves for the circulation of oil. When oil is forced between the plates at the normal working pressure the thrust bearing will carry the whole load. The weight of the rotor is about 32,000 lbs. The thrust bearing is of such dimensions that it may also be used with a reduced pressure of oil, in conjunction with an auxiliary hydraulic balance piston under the turbine runner. The balance piston is supported by a bridge-tree built into the draft tube, one arm of the bridge-tree being hollow to admit water under the pressure due to the operating head. The bridge-tree enlarges into a chamber, above which the runner flange rotates; this chamber is lined with a non-corrosive bronze gland and the runner hub, which revolves around it, is similarly lined. This arrangement does away with stuffing boxes, and the water leaking from the cylindrical surfaces escapes directly into the draft tubes.

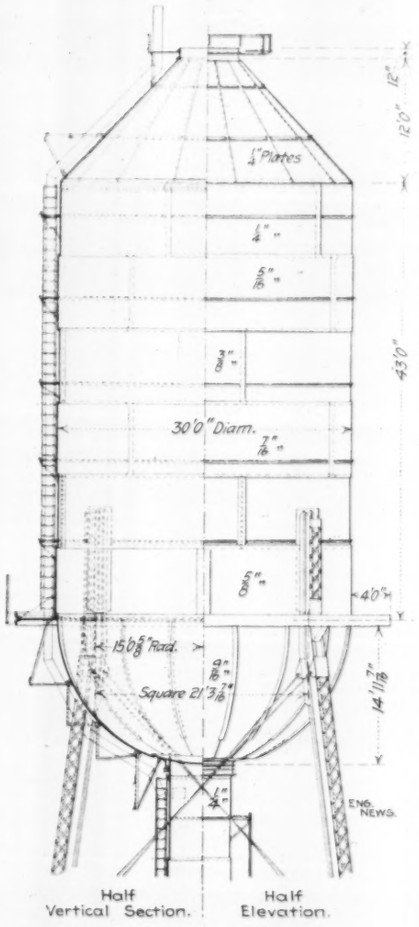
The hydraulic load-bearing materially reduces the consumption of oil and the power loss due to the operation of the pumps. This bearing enters into effect as soon as the penstock is filled with water and the gate valve is opened to the desired amount for carrying a part of the rotating weight.

Each generator turbine is closed off from the penstock by means of a 5½-ft. hydraulically operated gate valve. The exciter wheels are provided with gate valves operated by hand from the generator floor. Hand operation of the speed gates being impracticable on account of the great size of the units, independently driven oil pumps are provided to start up the turbine. Close to the exciter turbines are placed two high-pressure oil pumps, each driven by a separate impulse wheel. The oil under pressure is led to pressure tanks with which the exciter governors, the generator governors and their thrust bearings are connected by means of piping, valves, check valves and relief valves.

The exciter turbines are of the builders' downward-discharge impulse-wheel type, with vertical shafts. Each has a speed gate consisting of a simple pivoted tongue-vane connected to two pistons; one piston tends to close the vane by water under the pressure due to the operating head, the other acts under a variable oil pressure regulated by the governor. The apparatus, therefore, has always a tendency to close. The tail water is held at a certain level by means of an automatic valve. Upon the wheel case rests the thrust bearing, which operates in a bath fed with oil (not under pressure). A steady-bearing with a split gland lined with babbitt is attached to the thrust bearing. All the upper parts of this turbine are split so that they may be removed without disturbing the parts of the exciter. The governors are of the same construction as those for the main turbines, and are driven from the main shaft by means of bevel gears.

The hydraulic governors are of a special type, so designed that if desired the speed may be varied or the turbines started or shut down from the switchboard. They are also provided with an arrangement for regulating the degree of variation of one governor from the others. This is said to be specially advantageous when the power is to be used for different kinds of service at one time. The governors are provided with oil and water pressure gages which show the pressures on the parts controlled, which are in the basement. Each governor is operated from the main shaft of its turbine by means of bevel gears and a rigid drive.

The relays, which are controlled by the governors, are located on the spiral cases of the



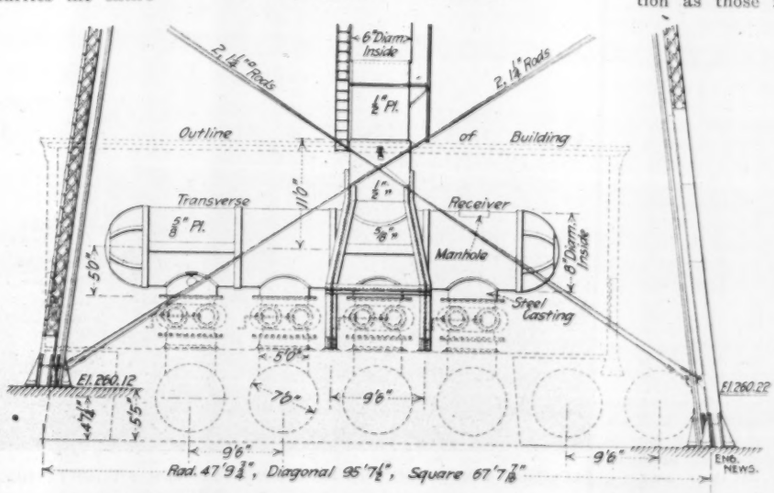


FIG. 8. DETAILS OF ELEVATED TANK, BASE OF SUPPORTING TOWER AND RECEIVER.

generator turbines and upon the inlets of the exciter turbines. This does away with the necessity for heavy shafts, arms and bearings for transmitting the power necessary for governing the turbines, and the power is transmitted only the shortest possible distance. The relays are constructed so that they have a constant tendency to close the turbines, thus going to the position of safety in case of accident to the oil pressure system. Each relay consists of two cylinders. On the generator turbines one cylinder communicates directly with the scroll case and is subject to a like water pressure, while the other is fed by a variable oil pressure. The oil pressure piston has a larger power than the water piston. If, for any reason, the oil pressure falls below a predetermined value, the turbine is closed by means of the self-acting water piston. The turbine, therefore, cannot run away because of a failure of the oil pressure.

The pressure regulating device is also operated by the governor, the connection to the governor being separate to prevent vibrations reaching the latter. The water cylinder of the relay has no valves and it is made of large dimensions to preclude any chance of its clogging. Each generator turbine is provided with two pressure regulating devices, and as they work in conjunction with the governor and are intended to pass off an amount of water corresponding to that which the turbine would have consumed in the next instant if the gate opening had not been closed, it is claimed that the two devices will approach nearer to ideal operation than one. One of these may be used as a reciprocally acting by-pass in case the condition of the penstock warrants it, while if one has to be repaired the other will suffice for the temporary service. By means of a special construction the device may be used to open only in the event of the gates being opened so suddenly that serious shocks would otherwise result. The discharge pipe of these devices is separate from the draft tube so as not to disturb the operation of the turbine or governor. Each device may be kept absolutely closed when desired by means of back pressure applied by a special gate valve.

A SIMPLE MODE OF ARRESTING SHORE EROSION.

G. C. SCHERER.*

From a number of experiments conducted by the War Department at different points along the Atlantic Coast, notably at Anastacia Island, Fla., South Island, S. C., Oak Island at the mouth of the Cape Fear River, N. C., and at Fort Macon, N. C., it has been found that the expensive Case system of groynes, used to arrest the erosion of the shore line where subjected to wave or current action, is not so satisfactory as a much more simple and inexpensive construction.

When these experiments were first undertaken, an ordinary sand fence was constructed, consisting of single posts of scantling set in the ground, 8 ft. c. to c., without any anchoring device at the bottom of the post. To these posts were nailed longitudinal boards. From the main fence, wings of varying lengths, of similar construction, were built at right angles on either side, the wings on one side being midway between the wings on the opposite side, the function of the wings being to limit the travel of sand along the main fence, and to enable the whole structure to catch sand in any direction.

These sand fences normally extend very near to the low water line, but serve more to arrest the sand transported by the wind than by the waves. When the first line of boards has arrested sufficient sand to about cover itself then an additional board is added, and this continues until that section of the beach which it is desired to rebuild has been sufficiently raised.

Further experiments along this line have been conducted with marked beneficial effect at Oak Island, at the mouth of the Cape Fear River, N. C., by building groynes of bags filled with beach sand, which bags of sand having no greater specific gravity than the sand upon which they rest, do not imbed of their own weight, but serve to catch and arrest the particles of sand in suspension during the ordinary prevailing winds of the seashore.

The first experiment which came under the writer's notice was on Anastacia Island, Fla., where the single line fence was used; that is, a fence without wings. Fig. 1 shows the condition of the beach at the time the fences were built; Fig. 2 shows its condition six months later.

Fig. 1. The Fence Just Built.

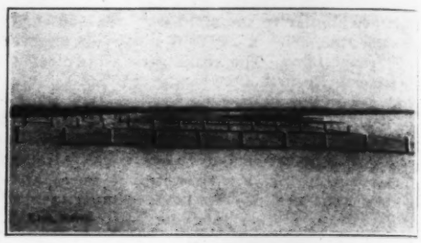


Fig. 2. Six Months Later.

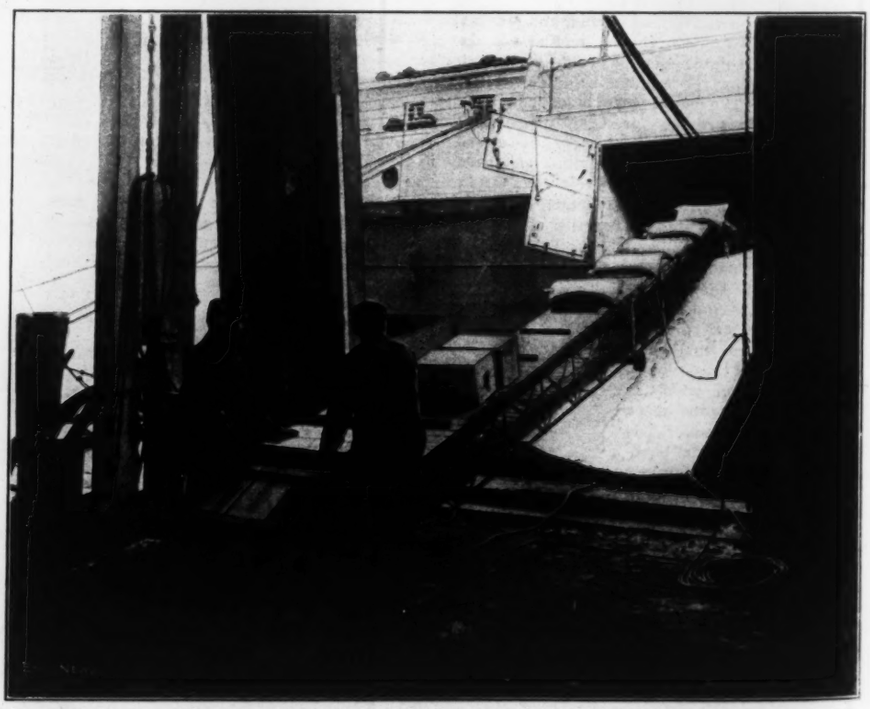
FIGS. 1 AND 2. ARRESTING SHORE EROSION AT ANASTASIA ISLAND, FLA., BY POST AND BOARD FENCES.

*U. S. Engineer Office, Wilmington, N. C.

A FREIGHT CONVEYOR FOR STEAMSHIP PIERS.

The handling of freight between a steamer and the pier is usually effected by means of hand trucks and steam hoists, but in some few cases conveyors have been applied to this class of service. The accompanying illustration represents an electric conveyor bridge used for this class of work. Two latticed trusses, 35 to 60 ft. long, carry the conveyor, which consists of an endless traveling platform (fitted with cleats) of wooden strips attached to a steel cable, which runs on steel sheaves having roller bearings. The motor and driving gear are mounted at the middle of the conveyor, between the trusses. The small wheels at the side are for use in moving the conveyor about the dock, it being handled in the same way as an ordinary gangway. The machine can be driven in either direction, to deliver packages to or from the vessel, and may be fitted with a counting device to register the number of packages handled. Horizontal extensions can convey the packages along the dock to any desired point.

The illustration shows one of two conveyors in use at the Old Dominion Steamship Co.'s pier at Norfolk, Va. They are also used at the piers of the Great Northern Ry., at Seattle, Wash., and the Northern Pacific Ry., at Duluth, Minn. Four were installed some time ago at the piers of the Tehuantepec National Ry. (Mexico), whose terminals were described in our issue of July 5, 1906. One of the latest plants is for the Federal Sugar Refining Co., of Yonkers, N. Y., to unload raw sugar (in 350-lb. sacks) from the steamers. This has a 50-ft. bridge, with 7-HP. motor, and also a 50-ft. extension, which is driven from the main conveyor by sprocket wheel and chain. The stationary conveyor may be set flush with the floor of a warehouse or freight house. These machines are built by the Spence Mfg. Co., of St. Paul, Minn.



ELECTRIC CONVEYOR FOR HANDLING STEAMER FREIGHT; OLD DOMINION STEAMSHIP CO., NORFOLK, VA.
Spence Mfg. Co., St. Paul, Minn., Builders.

RECENT STREET CAR ACCIDENTS in Chicago, by which persons have been seriously injured through being caught between passing cars, has led to another agitation to have the Board of Engineers reconsider its decision to use closely spaced tracks in connection with the new cars, which are 9 ft. wide.

WEATHER REPORTS BY WIRELESS TELEGRAPH are now being received by the U. S. Weather Bureau from the principal transatlantic ocean steamships. It has always been a difficulty of the Weather Bureau that no information was available as to atmospheric and weather conditions to the eastward of the Atlantic coast, and many of the failures of weather predictions in the Eastern States are to be ascribed to the lack of knowledge of the forecasters of the weather conditions over this area. The annual report of the Chief of the Weather Bureau states that during the past year weather reports to the number of 738 were received from vessels at sea. Of this number, 679 were received from transatlantic liners located between Sandy Hook and long. 44° west.

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The astonishing epidemic of great coal mine disasters, on which we have commented in this column for the past three weeks, was added to on Dec. 19, when an explosion took place in the Darr Mine of the Pittsburg Coal Co. at Jacobs Creek, Pa., in the Connellsville field, 40 miles south of Pittsburg. The number of men in the mine when the explosion occurred is believed to have been about 200 and every man inside was killed. There seems little room for doubt that the explosion was due to dust since the mine was equipped with a large ventilating system, which should have kept the workings free enough of gas to confine any gas explosion to a comparatively small part of the mine.

With the 34 miners killed at Naomi, Pa., on Dec. 1, the 350 at Monongah, W. Va., on Dec. 6, and the 60 at Yolande, Ala., on Dec. 16, this latest disaster makes the total casualties by coal mine explosions in the United States over 650 within three weeks, a record that should startle the American public to a realization of this country's disgraceful position regarding mine accidents.

On the very date of this latest coal mine horror at Jacobs Creek, the U. S. Geological Survey issued a bulletin on "Coal Mine Accidents; Their Cause and Prevention," which showed by incontrovertible statistics that the United States is not only far behind European nations in its provisions for the safety of coal miners, but that accidents in our coal mines are growing more frequent and serious. The comparisons of five leading nations is as follows:

NUMBER OF MEN KILLED IN COAL MINES ANNUALLY PER 1,000 MEN EMPLOYED.

Nation.	Years averaged.	Ave. men killed per 1,000 per annum.
United States	1901-1906	3.36
Belgium	1901-1906	1.02
Great Britain	1901-1906	1.29
Prussia	1901-1904	2.01
France	1901-1905	0.91

It is not a matter of haphazard that these European nations show fatalities which are on the average barely one-third those in the United States. Belgium in the years from 1851 to 1860 averaged 3.28 miners killed per annum per 1,000 men employed. She then established a bureau for the systematic investigation of mine casualties and has to-day the most thoroughly equipped mine testing station in the world. Her accident ratio has been steadily reduced since that time, until, as shown above, it is now less than one-third the former rate.

Not only have foreign countries obtained by scientific investigations the necessary knowledge with which to reduce the hazards of the coal mining industry, but they have made the application of such knowledge compulsory. In all the countries above mentioned except the United States, the use of excessive charges of explosives in mines is restricted, with the result of greatly reducing the casualties due to fal's of the roof, as well as the number of gas explosions. So too the use of safety lamps, the storage of explosives, the tamping of charges, the adoption of flameless explosives, the watering of passages to prevent dust explosions and many other precautions have been made compulsory by law, and observance of the law is made certain by a force of official inspectors.

In the light of the facts and statistics given in the Geological Survey report above referred to, it is clear that the United States is not less than a quarter century behind other civilized nations in its care for human lives at risk in the coal mining industry.

At this Christmas season, it may not be inopportune also to again refer to the fact that all the other leading nations require the industry itself to bear the burden of its casualties instead of laying this burden on the shoulders of the injured workmen or the bereaved families left by those who suffer death.

The coal mining industry is an admirable illustration of the crying need for reform in our employers' liability legislation. It will never be possible, with all known safeguards, to wholly eliminate the dangers which beset the coal miner, but after the state has done what it can to ensure safeguards, other civilized nations provide that compensation shall be made from the proceeds of the industry to the workmen who sacrifice lives and limbs in carrying it on.

This, be it noted, does not involve any serious burden upon industry. The number of men killed in the coal mines of the United States in 1906 per million tons of coal produced was 5.57. If we assume that three men suffered injuries for each person killed and assume also a rate of compensation of $3,000 for each man killed and $500 average compensation for injuries, we find that the total would amount to only 2½ cts. per ton of coal produced.

This is the manner in which other civilized nations provide for those who keep in motion the wheels of modern industry and expose themselves to its necessary risks. President Roosevelt has given the great weight of his recommendation that a similar system should be adopted in the United States.

If any one adheres to the belief that our American method is the better system we invite them to a perusal of the appeal just issued by the Monongah Miners' Relief Committee. We quote from it as follows:

> Fully 250 widows and 1,000 children are left without any means of support. As near as can be determined at this time almost $50,000 from all sources has been subscribed. Upon the basis of $300 for each widow, and $100 for each child under 16 years of age, $175,000 will be required. At least $25,000 additional will be needed to provide, on relatively the same basis, for the aged dependents of the unmarried victims, the many now unborn children and the undoubted accessions to the married victims' list when complete information has been received.
>
> The committee feels that a basis lower than the above will not be adequate to properly care for the needs of the bereaved ones, and a total of $200,000 will therefore be required to carry out the relief work on this basis.

Suppose the response to this appeal—made in these days when financial stress closes the pockets of many who ordinarily respond—proves inadequate? What about the sufferers in the three other great recent disasters? Is it not an inhuman absurdity in this democratic land that the whole burden of the risks of modern industry should be placed on those who are least able to bear it? Is it just to make paupers of the tens of thousands of workmen and their families who suffer every year by industrial accidents? Are there not abundance of other avenues where charity and generosity can do effectual work without laying on them the burdens which industry itself ought to bear?

We do not recall anywhere a precedent for the case which was on trial in the New York criminal courts last week. In February last, a derailment occurred at Woodlawn to one of the new electric trains of the New York Central & Hudson River R. R.; 24 persons were killed and a large number injured. The District Attorney's office finally fixed on the General Manager of the railway as the party criminally responsible for the wreck and he was placed on trial for manslaughter. The prosecution spent days in presenting the case, but wholly failed to show that the officer in question either did or failed to do anything to cause the wreck in question; and the Judge directed a verdict of not guilty to be rendered.

Substantially the same result was reached in this case as in the case nearly twenty years ago where the Directors of the New York, New Haven & Hartford R. R. were indicted as being responsible for the collision in the Park Ave. tunnel. In the more recent case, too, where directors of the street railway company operating in Newark, N. J., were indicted for the Clifton Ave. grade crossing disaster, acquittal resulted.

There is a popular clamor, after every serious disaster, that the law shall reach and punish the "man higher up," who is supposed to be ultimately responsible; but this popular clamor always subsides before the case is brought to trial; and sober public opinion would never approve the punishment for manslaughter of any man unless personal criminal negligence could be traced directly to him.

This being the case, it is opportune to ask whether the time of the courts and the public funds should be spent in trying such cases. The object in convicting and punishing a criminal, it must be remembered, is not revenge. Organized society seeks the punishment of crime for the purpose of warning other criminals against a repetition of the act. If the mere putting on trial for manslaughter of a Director or President or Manager, causes other officials of high rank to take greater care in the discharge of their duties, then the trial may be worth all it costs. If on the other hand the result of such trials is an impression that the law is inadequate to fix the responsibility for disasters and that officers of corporations are practically immune from punishment for negligence, then such a trial does far more harm than good.

Our own opinion is that this noted case has shown the impotence of the criminal court machinery to perform any useful end with relation to railway disasters. So far as bringing out the important lessons to be learned for the general benefit of the public and the greater safety of travel, the trial last week was as much of a farce as any coroner's jury investigation. This was not because of incompetence on the part of the prosecuting attorneys; but simply because they approach any such case from the wrong point of view, and without the expert knowledge that is absolutely essential to any useful work in the investigation of accidents.

The new Public Service Commission of New York can develop an expert bureau for the investigation of accidents which will be of real value in raising the standard of safety and improving practice; and it can do this at a trifling cost compared with the cumbrous and expensive machinery of the criminal courts.

Incidentally it may be remarked that while the cause of the accident still remains more or less of a mystery, and it is yet to be shown that any officer or employee was guilty of criminal negligence in connection with it, the railway corporation has acknowledged its responsibility by the payment of damages to those injured

in the wreck and the families of those killed, and according to reports has already paid out nearly a million dollars in this way.

In the Judge's statement ordering a verdict of acquittal, he referred to the testimony of Prof. Lovell, of Columbia University, an expert witness for the prosecution, as an abstruse and theoretical statement of the case and said:

It seems to me that the testimony of Prof. Lovell must give way to that of experienced and practical railroad men.

The impression is likely to go forth from this that the results of mathematical investigation of the behavior of railway vehicles on curves, contradict the experience of practical men. Such an impression needs correction. A true theory of "curve mechanics" will accord exactly with the results of the experience of practical men, when that experience is rightly understood and interpreted. We shall not take space here to discuss conflicts in the theory of this subject; but it is proper to say that if the defence had been permitted to present its side of the case, experts would have been placed on the stand to present a different theory of curve mechanics from that given by the expert for the prosecution.

The attention of every one of our readers interested in hydraulic power plants must certainly be attracted by the notable pressure regulating system described in this issue and in use at the Thomson, Minn., plant of the Great Northern Power Co. An elevated stand-pipe with a tank at the top rises from the receiver at the foot of the steel penstocks to a height of 235 ft.

The use of this stand-pipe was rendered necessary by the extraordinary length of the pipe lines supplying the power-house. These pipes are nearly a mile long and 7 ft. in diameter. With such long pipes the inertia of the mass of flowing water is so great that the ordinary method of water-wheel governing by shutting down the flow when the load is reduced and opening the supply valve when the load is increased, would produce exactly opposite speed variations from those desired. The mechanical reasons for this have been so fully explained in these columns that we need not further refer to them.

While such stand-pipes have been used in a few power plants of moderate size, the Thomson plant is by far the largest in which stand-pipe regulation has been adopted.

We believe that the stand-pipe system is worth adoption on plants with penstocks of moderate length as well as those where the conditions make its use compulsory. This is particularly the case where economy of water is an object, for other systems of pressure regulation in common use do not effect maximum economy in the use of water as does the stand-pipe.

There are, of course, plants at work under exceptionally high heads in the far West where an open stand-pipe would be too lofty to be practicable. Such plants can reach nearly as good results by the use of closed stand-pipes with their upper part filled with compressed air; but in any such installation it must be borne in mind that large volume is essential to the most thorough success. The compressed air will expand to follow up a drop in pressure even more promptly than the falling water in a vertical stand-pipe; but unless a large volume of compressed air is stored, the pressure will fall rapidly as expansion takes place. Similarly, the large volume is necessary to avoid too rapid rise in pressure when a supply valve to a wheel is throttled down.

In a compressed air pressure equalizing system a part of the necessary volume can be provided in a closed tank placed on the ground and connected to the upper part of the stand-pipe. Means must also be provided for keeping up the charge of air and regulating the level at which the water stands in the pipe. The open top stand-pipe, therefore, where heads are not too great, has much the advantage in simplicity.

NEW TESTS OF LARGE COMPRESSION MEMBERS: THE LIMITS OF COLUMN STRENGTH.

We are sure that every engineer interested in structural work will give long and careful study to the detailed report of compression tests on full size bridge members—chords and posts—printed in this issue. The tests were made and are reported upon by Mr. C. P. Buchanan, Bridge Engineer of the Pennsylvania Lines West of Pittsburg. It is the largest and most important series of tests on full size compression members for modern bridges that has ever been made.

It has often been pointed out that the working rules and formulas for column strength used by the engineering profession are chiefly based on experiments with small models, such as the classic tests of Hodgkinson. Every engineer to-day, we are safe in saying, feels the need of supplementing our knowledge of column strength by actual tests to destruction of large columns, exact duplicates of those in actual use.

Although these tests by Mr. Buchanan were made a number of years ago, they have never before been published; and in fact it was not until the Quebec Bridge disaster brought home to the profession the need of more definite knowledge of column strength that their importance was fully appreciated.

Mr. Buchanan's tests are notable not only for the size of the columns tested, but for the range of time over which they extend. The 19 tests extend over 14 years. Notwithstanding this, the series of tests is a coherent whole. All the tests were made by the same methods and on the same machine. The tests were made for a definite purpose: To develop methods of construction which would ensure a column stronger in its details than in the body, and at the same time give maximum crippling strength of the body. Table I. of the article exhibits this clearly. It shows that, after a few early tests of Z-bar columns, eight tests of chords and posts were made before failure in the end details was eliminated.

Further, and still more important, the tests are preeminently working tests, not laboratory tests. The columns experimented on were shop duplicates of actual chords and posts built and erected in bridges on the Panhandle lines. The tests had the object not of demonstrating a law of columns, or illustrating one or another type of strength or weakness, but of determining the actual strength of these members. The results, therefore, carry that force of conviction which distinguishes a service demonstration over a laboratory demonstration.

It is gratifying to observe that the practical character of these tests does not imply an absence of the consistency which we ordinarily expect only in careful laboratory experiments. Though the results as they appear in the tables seem to indicate a large range of variation, yet we learn from several pairs of identical or closely similar tests that a single result would duplicate itself surprisingly well in a second tests. Colums I. and II., III. and IV., X. and XI., and XVIII. and XIX., are such pairs of substantial duplicates; the crippling loads in each pair agree so closely that they fully establish each other's reliability.

The report of these tests, as we present it, is limited to a bare statement of the work done and the direct numerical observations. The author has purposely refrained from any analysis or comment, in order that the data might be available without prejudice to others for establishing their own deductions and conclusions. The study of what may be learned from the tests in the way of proper construction in detail, influence of plating and latticing, or variation of column strength with length-ratio, etc., is left to the reader.

There is, however, one broad result which speaks so plainly from the figures exhibited in Mr. Buchanan's records, that a few words concerning it seem advisable.

We refer to the bearing of these test results on the safe working compression stresses to be adopted by engineers. It need hardly be said here that the Quebec Bridge failure is [illegible] responsible for bringing this matter of safe [illegible]working stresses to the front.

The designers of the Quebec Bridge [illegible]lited the compression members with a u[illegible]zable strength of 24,000 lbs. per sq. in., an u[illegible]ually high working value. It has been general[illegible] concluded that the failure of some of thes[illegible] members was due to weakness of assemblage, [illegible]com-pound of several single weaknesses in [illegible]tail construction. This conclusion has, h[illegible]ever, still left in the minds of many engineers [illegible] belief that the working stress was too h[illegible] in any case, that it would have spelled d[illegible]nger even if the columns had been free fro[illegible] any weakness of assemblage. Yet there have been no authoritative test-figures that could [illegible] referred to as proof or disproof of this view.

Many of the formulas for column resistance in use even to-day are based on ultimate tensile strength, although the substitution of the elastic limit or yield point has long been advocated. It is well known that when a tension member is stretched beyond its yield point it is permanently injured and must be replaced, hence all estimates of working strength must start from this value. The yield point of the material is even more important in the case of columns, since it is influential in determining the crippling load of the column. A suggestive statement on this subject was made over 20 years ago before the American Society of Civil Engineers by Mr. C. C. Schneider.* We quote from it as follows:

I fail to see the reason why Mr. Wilson follows the older specifications, and specifies a smaller unit strain for compression than for tension. All experiments prove that the ultimate strength, as well as the elastic limit, are practically the same for compression as for tension; why, then, not allow the same working strain in both cases? The usual explanation given by engineers who adhere to this practice is: The material in compression members buckles or cripples before rupture takes place; therefore, for compression members we take this buckling strength as a guide to determine the permissible unit stress while for tension members we take the ultimate strength. A good rule, however, should work both ways. The tension members are also crippled before rupture takes place; and in this respect all members are alike. Compression as well as tension members, are practically crippled as soon as their fiber strains exceed the elastic limit of the material. I think we should be guided by the elastic limit of the material in determining the permissible stresses more than by the ultimate or buckling strength, as in practice we do not intend to reach either, but we want to keep enough below the elastic limit to leave a fair margin for defects in the material and for ignorance.

This was an advanced view in 1886. Since then, it has become general to regard the yield point of the material and not the ultimate strength in fixing the safe load of columns. Yet even to-day the older practice has not wholly disappeared. The most recent edition (1907) of Johnson, Bryan & Turneaure's "Theory and Practice of Modern Framed Structures," presents an argument against the older practice in opening the chapter on columns:

For semi-plastic materials, like the rolled metals, the elastic limit is, or should be regarded as, the ultimate strength. . . . In all grades of rolled iron and steel the elastic limit in compression is practically identical with this limit in tension, and hence the "elastic limit," as found by a tension test of the material, may be regarded as the ultimate strength of that material in compression.

Now, nowhere in this passage or in the remarks of Mr. Schneider above quoted, or anywhere else in engineering literature so far as we recall, is it suggested that the crippling strength of a compressive member—the point where its final failure occurs, may be *even lower* than the yield point of the material. We do not now refer to the reduction properly to be expected in long columns, as expressed by every column formula, but to a difference between the yield point and the base constant which serves as the initial point of the column formula. There are theoretical reasons for expecting such a discrepancy, for example the non-uniformity of distribution of centric stress due to initial stress in parts of the column, or the influence of rivet-holes, etc. Yet no suggestion has been made that such an effect may occur, and no one has uttered a caution on this account in fixing column stresses and margins of safety. Any discrepancy of the sort is very important, however, since it eats very rapidly into the supposed margin of safety.

*In discussion of A. M. Wilson's paper "Specifications for the Strength of Iron Bridges," Trans. Am. Soc. C. E., XV. (1886), 482.

The question whether such a discrepancy does occur can only be answered by actual test of full-size columns, and Mr. Buchanan's tests representing as they do compression members actually used in bridges are an authoritative answer. A glance at the results shown in Mr. Buchanan's Table I. shows that in every test *the crippling load is considerably below the yield point of the material.*

The iron and steel used had a yield point of from 30,000 to 34,000 for iron and up to nearly 40,000 lbs. for the steel. The crippling strength of the large columns tested, however, ranged only from 24,000 lbs. to 34,000 lbs. per sq. in., even though the series included columns so short that the "column reduction" effect is virtually eliminated.

The very first column of the series, tested in 1888-90, two of iron and two of steel, failed at an average value of 25,500 lbs. per sq. in. for the iron and 33,500 lbs. for the steel; in other words, some 5,000 lbs. below the yield point of the material. These, it is true, were relatively long columns. Their failure value may well be supposed to have been lowered by the column-reduction factor. The later columns tested, however, are nearly all of much lower length-ratio, ranging well below 50. These may be referred to with more confidence. They are "short columns." Many authorities specify no column reduction for length-ratio less than 40 or 50, on the view that such short members develop practically the full, unreduced column resistance of the material, and the shape of the curve of the Rankine-Gordon formula (which is generally accepted as a reliable representation of the column law) supports this view. Yet, on referring to Mr. Buchanan's table, it is seen that these short columns give results little different from those obtained with the longer columns. Their failure strengths are from 5,000 to 8,000 lbs. per sq. in. below the yield point of the material.

What causes are responsible for the relatively high strength of the long columns of the series, it is not possible to investigate here. The four long columns tested in 1888-90 were of a different type from the shorter ones, and this may have bearing on the matter; in fact, the fifth long column, of latticed type; shows a much reduced strength. Moreover, many of the short columns were limited in strength by failure in specific details, though the table does not indicate this to have caused any considerable loss in strength.

But we must recognize that, irrespective of type or details, all the columns tested represent everyday commercial structures, which by universal practice are proportioned under identical rules and therefore are credited with identical ultimate strength. The question before us is to fix the amount of this limiting strength, from the data afforded by the new series of tests. Later studies may show that if we adopt one certain form of column we are justified in reckoning with a higher strength than the average, or that, if every possible failure due to weak details be adequately guarded against, a higher strength limit than shown by these tests will be realized. But in the present state of knowledge and practice in design we must group all standard types of column together. The U-sections of Mr. Buchanan's chords, and the latticed sections of his posts, are standard forms, and are everywhere designed by the same stresses and column formulas as any other type, say the Z-bar or the box column.

As already noted, the "short columns" of the series are so near together in length-ratio (29 to 46) that they may be taken as parallel cases and may be averaged together. The "long" columns of the series (length-ratio 83 to 120) should be expected to show lower strength. The results are to be compared with the values given by Mr. Buchanan for the average yield point of the material, 30,700 lbs. per sq. in. for the iron and 39,900 lbs. per sq. in. for the steel. Such a comparison is given in the table of crippling loads in the next column.

The percentage figures given for wrought iron are affected by the fact that the elastic limit of the material varied considerably, the angles often being 3,000 to 5,000 lbs. higher in yield point than the plates. As the flange material doubtless influences the result more than the web, it is possible that the observed results should be referred to a higher yield point than 30,700. This would tend to bring the percentages into better agreement with those figured for the steel columns.

CRIPPLING LOADS.

Wrought-iron columns.	Lbs. per Sq. In. Range.	Lbs. per Sq. In. Average.	Per cent of Yield Point. Range.	Per cent of Yield Point. Average.
All (12)	23,300-30,800	27,500	76-100	89
Body-failures only (5)	24,800-30,300	27,200	81- 99	89
Short columns only (10)	23,300-30,800	27,900	76-100	91
Short columns, body failures only (3)	26,260-30,300	28,250	85- 99	92
Steel columns:				
All (7) [all body failures]	27,800-34,300	31,900	70- 86	80
Short columns only (4)	30,600-34,000	32,050	77- 85	80

Taking the figures as they are, however, two things are clearly apparent: First, that the crippling loads reached only 90% of the yield point in the wrought-iron columns and only 80% in the steel columns; second, that the "short" columns show no gain over the average. No rising tendency of the strength seems to accompany the low length-ratios, and it is reasonable to conclude that the results represent the limiting strength of such columns irrespective of any "column reduction."

The general result may therefore be expressed thus: Well-made wrought-iron or steel columns fail completely at loads which on the most favorable assumption will not exceed 90% of the tensile yield point of the material. A steel column whose material shows 40,000 lbs. tensile yield point will, when loaded to 18,000 lbs. per sq. in., have a factor of safety of not over 2, even if its construction be of the best type. In contrast therewith, a tension member of the same material, loaded to 20,000 lbs. per sq. in., has a factor of safety against failure exceeding 3, while against permanent deformation it has a factor of 2, which is as great as the factor of safety of the column against final collapse.

This is the most important showing of the tests recorded. It is a most impressive warning of the danger of that gradual increase in working stresses which has been quietly going on for eight or ten years past. In column design particularly, we are warned to return to more conservative practice.

One other vitally important result of Mr. Buchanan's test must be noted. This is the appearance of an unmistakable yield point, or elastic limit, of the columns as a whole, and the amount of this limit, as indicated by the deformation curves.

The limit of elastic action of the columns tested is not so clearly fixed by the curves that unimpeachable values can be read from them. The author of the report has fixed an "Elastic Limit" and a "Yield Point" for each column, from the curve of total compression. But we may also consider the limit of elastic behavior to lie at the point where permanent set begins, or (since the deformation of the pin ends is included in the record) at the point where a pronounced change in the set curve occurs. On the other hand a designer may prefer to be guided by the deflection records, since in a column which fails by bending we must needs identify the beginning of increased bending with incipient failure. On this third view, we would select the point where one of the deflection curves (or strictly speaking their resultant) begins to flatten its slope. It is thus possible to read the desired value in at least three ways.

In the curves shown, the three values thus obtained for the limit of elastic behavior are by no means concordant. The table in the next column gives our estimate of the three values; the value derived from the curve of total compression occasionally differs from both Elastic Limit and the Yield Point marked by Mr. Buchanan.

These tabulated figures, as well as the curves from which they are derived, make clear the fact that in the upper part of the range of column resistance there is a domain of imperfect elasticity, just as there is a similar domain in the upper range of tensile resistance. In tension tests this domain covers something more than the upper one-third of the ultimate—that is, the elastic range is not quite two-thirds of the ultimate strength. Judging from the above table the domain of elastic behavior in column resistance extends little higher than two-thirds the collapsing strength.

LIMIT OF ELASTIC BEHAVIOR, COLUMNS V-XIX.

Col. No.	From total compression curve.	From set curve.	From deflection curves.	Average.
V	15,000	12,000	18,000	I 15,000
VI	17,000	15,000	.....	I 16,000
VII	20,000	21,000	26,000	I 22,000
VIII	17,500	14,000	22,000	I 18,000
IX	14,000	12,000	12,000	S 13,000
X	20,000	14,000	21,000	I 18,000
XI	20,000	16,000	20,000	I 19,000
XII	22,000	24,000	29,000	I 25,000
XIII	22,000	18,000	21,000	I 17,000
XIV	21,000	21,000	22,000	I 21,000
XV	24,000	17,000	23,000	I 21,000
XVI	27,000	27,000	24,000	S 26,000
XVII	25,000	24,000	27,000	S 25,000
XVIII	26,500	25,000	25,000	S 25,000
XIX	24,000	18,000	31,000	S 24,000

Average iron, 19,200
Average, steel, 23,000

In conservative designing the intensities of permanent load and of repetitive loading must in all cases remain below the limit of elastic behavior. If this consideration be allowed to govern in fixing column stresses, we must take our limit not merely at .80 to .90 of the tensile yield point, as shown by the figures of crippling strength, but at .70 of this reduced value, or, say at 60 per cent. of the tensile yield point. We are then limited to about 21,000 to 24,000 pounds per square inch in steel, for the figures to be taken as the initial point in column calculations. Column reduction and the necessary margin of safety against uncertainties, imperfections, ignorance and service contingencies, will of course bring the practical working stresses to a value much lower.

LETTERS TO THE EDITOR.

Copper-Clad Steel Wire for High Tension Transmission Lines.

Sir: I notice in the article on the "Maintenance and Use of Copper Clad Steel," in your issue of Nov. 11, that the use of such wire, having a steel core, is suggested for high tension transmission lines. Evidently sufficient consideration was not given to the inductive effect of the steel core which would be so great as to prohibit the use of this conductor for such purposes.

Yours truly,

Howard W. Starr.

11 Front St., Schenectady, N. Y.

[The increase in energy losses, in apparent resistance and in drop of voltage of an alternating-current transmission line over a direct-current system are self and mutual induction, hysteresis, eddy current and static effects. The induction and hysteresis losses and disturbances will be the only phenomena at all appreciably affected by the use of a steel-cored conductor. These added losses and disturbances depend directly on the frequency and inversely on the separation of conductors. If we conceive a system of very low frequency and such high voltage that the conductors are very widely separated, then under some line conditions it may be considered economical to use the wire mentioned in this article for certain long spans on account of its greater strength. The electrical effects of such use need to be determined for any particular case and carefully considered before installation.

In this connection the practice may be noted of using ordinary "bi-metallic" wire (steel with a non-adhesive copper envelope) in some parts of California for the multitude of small branch lines, to irrigation pumping plants, from the trunk transmission lines. Here the loads are light or intermittent and it is desirable to keep the capital investment at a minimum for such service.

However, it is indicated, in the article mentioned in the above letter, that the chief and immediate use of this wire is for signal work rather than for ordinary high-tension, short span, alternating-current transmission lines.—Ed.]

A Water-Works System for Jerusalem.

Sir: It may be of interest to your readers to know that Jerusalem is likely to have a modern system of water-works. The matter is being seriously entertained, and Mr. E. F. Baldwin, one of the leaders of the American colony there, on behalf of the governor of the province, has written to me, requesting a report and estimate of the probable cost.

He gives data of several possible sources of supply. Being somewhat familiar with the topography and other conditions, because of a sojourn of two weeks in and about Jerusalem, three years ago, I have been able to comply with his request.

The estimate of cost for a supply of one to two million gallons per day, from the nearest source, is a little less than half a million dollars; from the furthest source, about three-quarters of a million. These would both be pumping systems under high pressure. The data given relative to other sources of supply were insufficient and I could make no estimate from them.

I recognize the nearest source as that of the fountain head of the brook Cherith, mentioned in Chapter 17 of I. Kings. It is upon this brook that Elijah is alleged to have dwelt and been fed by the ravens. Elijah had prophesied a drouth and famine. The account says:

(Verse 5) So he went and did according to the word of the Lord: for he went and dwelt by the brook Cherith, that is before Jordan.

(Verse 6) And the ravens brought him bread and flesh in the morning, and bread and flesh in the evening; and he drank of the brook.

(Verse 7) And it came to pass after a while that the brook dried up, because there had been no rain in the land.

Elijah then moved, according to the account which follows, and went to live with a widow at Zarepath, and notwithstanding she had her own family and one more mouth to feed, the amount of oil in her cruse and meal in her barrel never diminished.

This chapter and those which follow, although full of miracles and other nonsense, may be an authentic historical record of an event of practical value for reference at this time. If that spring went dry 2,800 years ago, it may go dry next year, which would be embarrassing to the designer and projectors of the works and to the inhabitants of the city who were depending upon it for water. In my report, I have called the attention of the governor to this passage of scripture, and advised him to try to get something more recent bearing upon the case.

Jerusalem is situated near the summit of the ridge between the Mediterranean and the Dead Sea. It is about 2,600 ft. above the former and about 3,900 ft. above the latter, and has a population of about 30,000. The country is rough and broken, especially on the Dead Sea slope. Rains are copious at times, but do not extend throughout the year, and the country is therefore subject to drouth.

The foreign colonies are numerous and are growing rapidly. They are situated outside the walls, and cover considerable territory. Being supported by governments and missionary and other societies, they are able to pay a good price for water.

Very truly yours,

F. W. Blackford,
M. Am. Soc. C. E.

Prolongacion del Cinco de Mayo Numero 9, Mexico, D. F., Dec. 2, 1907.

Rivet Slip as Limiting the Strength of Column Lattice.

Sir: In Engineering News of Oct 3, 1907, Mr. H. S. Prichard and Mr. John Bartholomew independently make note of the fact that with double systems of latticing and cross-ties in compression members the possible stress in the lattice members is a function of the stress in the main member. In chord A 9 L of the Quebec bridge I find the proportion of such stress to be 28.4%.

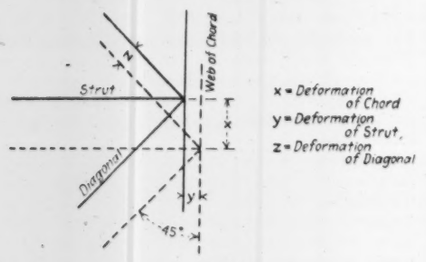

Fig. 1.

In Fig. 1 is shown a diagram of this effect. The relation between the sections of the cross struts and the diagonal ties is such that $y = 1.078\,z$, from which $x = 2.478\,z$. Allowing for its greater length, the diagonal will receive a fiber-stress of $(54 \times 15{,}000) \div (76 \times 2.478) = 4{,}200$ lbs. per sq. in., when the stress in the main member is 15,000 lbs. per sq. in. The total stress in the diagonal then is $4{,}200 \times 2.48 = 10{,}565$ lbs.

If this compression chord were bent, or imperfections in the manufacture caused an eccentricity such that when loaded up to a stress of 15,000 lbs. per sq. in. it had an inward bend of ½-in. with points of contraflexure at the quarter points, the total angular change over a length, of 28 ft. 6 ins. would be 40′. Assuming that there are five panels of latticing and that the bending is circular, the bending per panel would be 8′. The normal component of the stress in one panel of the main member, acting at an angle of 8′ with the next, would be the increment of shear per panel.

Log of 15,000 × 781	= 7.068742
Log sin of 8′	= 7.366816
Log	= 4.435558
Number	= 27,262

The total shear in the panels either side of the points of contraflexure would be $2 \times 27{,}262 = 54{,}524$ and, dividing over four lattice members and multiplying by the secant of 45°, each lattice diagonal would receive a stress of 19,083 lbs. In the second panel on either side of the points of contraflexure the stress would be 9,541 lbs. The stress in the diagonal due to direct load on the chord should be added to this for the compression members, giving 29,648 lbs. and 20,106 lbs. in the first and second panels, respectively.

In the Quebec bridge chords, to which these calculations apply, the lattice diagonals were connected to the main member by two ⅞-in. rivets at each end. Experiments by M. Dupuy, quoted in Johnson's materials of construction, give 12,000 lbs. per sq. in. of rivet section as the frictional resistance of iron rivets driven with a hydraulic riveter. Allowing for the increased strength of steel rivets, 15,000 lbs. per sq. in. should be a generous value. The two-rivet joint should therefore slip under a stress of $15{,}000 \times 2 \times 0.6 = 18{,}000$ lbs.

Therefore, eight of the compression lattices of this chord, two on either side of each point of contraflexure, would have slip. The actual amount of this slip it is impossible to determine without accurate knowledge of the conditions of driving the rivets and making the holes. Assuming, however, that the driving temperature was 1,000° F., and that Poisson's ratio applies over the whole period of cooling, we have $1{,}000 \times .0000065 \times 1¼ = .0081$. For two joints the yielding would be .0162-in.

The yielding of the compression lattice would cause a deformation of the square composed of two webs and two cross struts. This deformation would cause increase of tension in the tension lattice and decrease of compression in the compression lattice of equal amounts. That is, due to the elongation of the compression lattice due to relief of its stress, the tangent of the angular change would be $(½ \times .016 \div .7 = .0114) \div 54 = .00021$, giving an angle of 45″.

The rivets should slip the full amount since the friction of motion is about half that of rest, and unless the stress in the compression lattice fell below 9,000 lbs. it should not come to rest until bending of the rivet begins.

The relief of stress due to slip in the compression member $= S = (.0162 \times 2.48 \times 30{,}000{,}000) \div (2 \times 76) = 8{,}000$ lbs. The remaining stress in this lattice then equals $10{,}565 + 19{,}083 - 8{,}000 = 21{,}648$ lbs. In the second panel it is $10{,}565 + 9{,}541 - 8{,}000 = 12{,}106$ lbs. The tension lattice in the first panel would receive $10{,}565 - 19{,}083 - 8{,}000 = -16{,}518$ lbs. Now, the slip of the compression member making an angular bend of 45″ in two panels, plus the slight deflection due to additional stress from this bending, gives a total additional stress in this tension lattice of over 1,500 lbs. This brings its stress over the slip point and we have eight compression lattices and four tension lattices with slip.

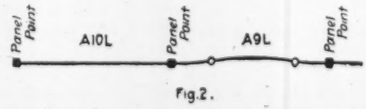

Fig. 2.

In Fig. 2 is shown the condition of bending observed in chord A 9 L of the Quebec bridge three days before the fall. The small circles mark points of contraflexure, whose location I have assumed as the quarter points. I have started with a center deflection of ½-in. for the half chord, or 1 in. for the whole chord. The increase to 1½ ins. or 2 ins. would have come from the slip of the twelve lattice connections.

Now either A 10 L or A 8 L might have a tendency to inward bend which the increased fiber stress has just developed. A bearing splice might have slip in its rivets and come to bearing, for instance. In Fig. 3 is shown the conditions of bending that would then exist.

Fig. 3.

The points of contraflexure have been shifted. Supposing them to have moved a distance equal to two panels, four more tension diagonals and four more compression diagonals would have slipped.

There are left at most in this chord two lattice members which have not slipped, and it is likely that the increased bending will have caused them to [illegible] at this stage.

The 45″ bending per panel for each slipped [illegible]mber is no longer the limit. As a rivet passes the [illegible]p point, its deformation for increase of stress becomes [illegible]rge. The slip of the tension lattice adjacent to the [illegible] of contraflexure will throw 8,000 lbs. back into the [illegible]pression diagonal, which, yielding further, will great[illegible] increase the angle of bend. With all or practically [illegible] of the lattices slipped and some of the rivets bendi[illegible] the deflection of the chord has reached a point [illegible]ere the ultimate strength of the rivets is insufficient [illegible] hold the webs together. Failing the chord would take [illegible]e form of the letter S as noted on the ground.

The frictional strength of the joints would [illegible] a limiting point in the strength of all chords laced in this manner unless gusset plates were used. The inefficiency of a bracing for a compression member depends on its stiffness rather than on its ultimate strength.

The deflection that these webs endured before failure points to some yielding other than that of ultimate strength of the bracing, since complete failure would have been caused before or for less deflection if all the deflection were caused by the extension or compression of the lattices. If the system were rigid and comparatively unyielding it would throw high eccentric stress into the webs, and failure would occur for less deflection though the fiber-stress were much higher. It seems to me that we are justified in having complete confidence in our compression formulas, even though they are derived from experiments on much smaller columns than this.

Albert Smith.

Purdue University, Lafayette, Ind., Nov. 19, 1907.

THE 80-CT. GAS RATE for Manhattan Borough, New York City, has been declared confiscatory and therefore unconstitutional by the U. S. Circuit Court.

APPRAISAL OF RAILWAY PROPERTY is to be made in South Dakota under direction of the State Railroad Commission, and Carl C. Witt has been appointed expert engineer in charge of the work.

DEAD SPARROWS IN A WATER TANK at Walden, N. Y., are the latest newspaper cause for a typhoid outbreak, but the same ridiculous story has been applied elsewhere before.

A TYPHOID OUTBREAK AT TRENTON, N. J., which has reached about 70 cases in a short time, is attributed to polluted Delaware River water.

THE FOURTH SERIOUS MINE EXPLOSION since December began occurred on Dec. 19 in the Darr mine of the Pittsburg Coal Co., near Jacobs Creek, about 18 miles west of Connellsville, Pa. Between 200 and 250 men were in the mine at the time of the explosion and it is believed that all were killed. On account of a church holiday many of the miners were not at work and so escaped death. All the circumstances point to a general dust explosion extending throughout the mine as the probable course of the calamity.

A HOISTING CABLE broke recently at the shaft of the Toronto water-works tunnel and a cage with seven men fell to the bottom of the shaft. The circumstances are described as follows by "The Engineering Journal of Canada":

The cable, we understand, was ⅞-in. in diameter, and was comparatively new, having been but a week in use. At the time of the accident the cage in which the seven unfortunate men were being hoisted was within 20 ft. of the surface. Just above their heads was a 500-lb. balancing weight, in which was inserted a sheave. The cable, fastened to the end of the boom overhead, passed through the sheave at the right, returning to pass over another sheave near the end of the boom, and underneath a third at the foot of the mast, and was attached to the drum of the hoisting engine, we presume, in the usual manner. The breakage was a clean one, little or no stranding having taken place, and was close to the drum.

Theoretically, the cable was supporting a weight very much less than it was equal to. A ⅞-in. steel cable is supposed to be good for a breaking strain of 22 gross tons, i. e., 49,280 lbs. The cage which was being hoisted would not weigh more than 400 or 500 lbs., and the seven men, an additional 1,200; that is, there was an approximate total of 1,700 lbs. suspended from the cable over the shaft. Of this, not more than 800 lbs. would be the stress on the cable at the point of breakage.

The cable, it is claimed, was of good material, having been manufactured by a reputable English firm.

Assuming the facts to be as above stated, the probability is that the breakage was due to the use of sheaves of too small diameter for the size and stiffness of the rope. No engineering investigation of the disaster was held and the coroner's jury brought in a verdict of "accidental death."

COMPETITIVE PLANS FOR A MUNICIPAL OFFICE building in connection with added terminal facilities for the Manhattan end of the Brooklyn Bridge are to be submitted to a jury of award by April 15, 1908, and an award is to be made by May 14, 1907. Mayor McClellan has

...ssed the following New York architects or firms of ...ects to enter the competition:

...Mead & White, Carrere & Hastings, Henry ... Marshall, J. Stewart Barney, Clinton & Russell, ... Boring & Tilton, Kreidlander, Hines & La Farge, Helme & Huberty, ... Hornbostel & Koen, Howells & Stokes, Trowbridge & Liv... ... Ludlow and Warren & Wetmore.

...Each competitor and each of the three judges is to be paid $1,000 for his services. The proposed building will occupy the two blocks bounded by Park Row, Centre and Duane Sts. A building 20 stories high, with 600,000 sq. ft. of office space, and costing $7,000,000, is proposed.

THE HUDSON MEMORIAL BRIDGE DESIGN, which was illustrated and described in our issue of Nov. 21, 1907, received the approval of the Art Commission of the City of New York at its meeting of Dec. 10, 1907. It is understood that the architectural features are to be worked out in greater detail before specific approval for these will be granted.

THE BUILDING ORDINANCES OF CHICAGO are to be thoroughly revised by a special commission, by the advice of the present Building Commissioner, Mr. Joseph Downey. The mayor will appoint a commission of seven experts to undertake the work. The ordinances as they now stand were hastily revised during the excitement following the Iroquois Theater fire, and have been found to be unsatisfactory and impracticable in many ways.

A PROPOSED NEW BUILDING CODE for New York City has been temporarily shelved by the Board of Aldermen. This code was prepared by an expert commission of some twenty members under the direction of the Building Committee of the Board of Aldermen, and was reported by the Committee to the Board two weeks ago. In its general character the code is identical with the existing code, but the sections have been rewritten and rearranged quite extensively. The principal points of novelty in the proposed code are: Limitation of height of buildings; a complete new section dealing with reinforced-concrete constructions, and a clause limiting to 85 ft. the height of buildings having walls, columns or girders of reinforced concrete; a more rigid and extremely specific section defining fireproofing, which incidentally requires that all fireproof buildings of the first class must have steel floorbeams spaced not over 5 ft. on centers and arched floor construction between the beams; and a more comprehensive classification of buildings as to fireproof character. Strong objection was made to the proposed code on the ground that it is discriminative against concrete, and that the fireproofing provisions are improperly drawn. The Board of Aldermen has referred the code to a committee and has adjourned to Dec. 31. As a new Board of different political complexion comes into office with the new year, it is thought probable that nothing further will be done with the pending code.

A VERDICT OF NOT GUILTY was brought in on Dec. 18 in the case of Alfred H. Smith, Vice-President and General Manager of the New York Central & Hudson River R. R., who was indicted for manslaughter in the second degree on account of criminal negligence in his management of the railroad, as a specific result of which negligence the Brewster express out of New York City was wrecked at Woodlawn, N. Y., on the night of Feb. 16, 1906, and 24 persons killed and many more injured.

This is the accident in which a train of the N. Y. C. & H. R. R. R., drawn by two electric locomotives, was derailed while rounding a 3° curve, superelevation of 4½ ins., with 100-lb. rails single spiked to ties every 20 ins., at a speed estimated at from 55 to 70 miles per hour. The circumstances surrounding the wreck and the subsequent investigations were quite fully treated in Engineering News, Feb. 21, 1907, p. 220; March 7, 1907, pp. 296, 275, and March 14, 1907, p. 297.

The prosecution based its case upon the claim that with the given track conditions the safe speed of the wrecked train around the curve was about 57 miles per hour, that the speed of the train upon the night of the wreck was in excess of this amount and was therefore such as to invite accident and that the General Manager of the road, as the supervisory head who regulated all details of track construction and speed limits, being responsible for the proper consideration of all these conditions which joined together to cause death and injury, was criminally negligent in allowing them to exist.

The Judge took the case out of the jury's hands at the end of the prosecution's testimony and directed a verdict of acquittal on the grounds that the charge was not proven. Although much technical testimony had been given, this verdict was rendered largely on the basis that a General Manager, having under his supervision 50,000 men and 7,000 miles of track, could not be expected to be personally responsible or criminally liable for the omission of proper precautionary measures at any one point along his line, provided that he had given general or specific instructions to his subordinates to safeguard the lives of the passengers. While admitting that the accident was probably caused by excessive speed, the Judge said that according to the evidence, the engineers had built the curve as safe as the state of the art required.

The principal evidence against the safety of the track system at the curve was given by Prof. E. B. Lovell, of Columbia University, who repeated his previous testimony before the coroner's jury and the State Railroad Commission, to the effect that the outward pressure of the moving locomotive on the outer rail of the curve was sufficient, at the speed at which the train was traveling, to shear off the outside holding spikes and cause derailment.

This theory is founded on the following combination of conditions and events which tend to concentrate the entire outward thrust on one spike and to shear it. First, the entire thrust is assumed to pass through the outer driver; second, the rail is supposed to be of such flexibility that it cannot transmit any of an applied load a distance of 20 ins. to the next tie, and third, that the tie under the rail at the point of shearing bend sunk to such an extent that no friction can exist between the tie and the rail.

No other mathematical evidence was produced; but the justice stated that Prof. Lovell's theoretical investigations were overweighed by the testimony of the New York Central Engineers, called by the prosecution, who testified that the track was amply safe for any train that had to or did go over it.

HEAVY PASSENGER LOCOMOTIVES of the ten-wheel or 4-6-0 type have been built by the Baldwin Locomotive Works for service on the Southern Pacific Ry.'s division between Sacramento, Cal., and Sparks, Nev., 158 miles. For the first 22 miles the grades are comparatively easy, and one engine can take a 500-ton train at an average speed of 27 m. p. hr. On the next 30 miles the maximum grade is 2% and a pusher engine will be required for an average speed of 21 m. p. hr. On the next 53 miles, the maximum grade is 2.25%, and here two of the ten-wheel engines will be used to give an average speed of 17 m. p. hr. The remainder of the distance has easy grades. The engines have large boilers of the wagon-top type, but the fireboxes are narrow, burning oil fuel. The tender has a cylindrical tank. The principal dimensions of these engines are as follows:

Driving wheels	5 ft. 3 ins.
Truck wheels	2 ft. 6½ ins.
Wheelbase, Driving	13 ft. 10 ins.
Truck	6 ft. 4 ins.
Total engine	25 ft. 10 ins.
Engine and tender	58 ft. 10 ins.
Weight, On drivers	159,750 lbs.
Engine	203,050 lbs.
Engine and tender	344,000 lbs.
Cylinders	22 x 28 ins.
Valves	Piston, 12 ins. diam.
Valve-gear	Stephenson link
Boiler; diameter at first ring	6 ft. 0 ins.
Steam pressure	190 lbs.
Firebox; radial stay; size	10 ft. 4 ins. x 3 ft. 1½ ins.
Tubes; No., 355; outside diameter	2 ins.
Tubes, length	15 ft. 0 ins.
Heating surface; tubes	2,788 sq. ft.
Firebox	206 sq. ft.
Total	2,994 sq. ft.
Grate area	32 sq. ft.
Oil in tender	2,940 gals.
Water in tender	7,000 gals.
Tractive effort	34,740 lbs.

THE SERVICE VALUE OF LOCOMOTIVE FUEL may differ considerably from its value by calorimeter test, according to a statement by Mr. L. E. Endsley in a discussion of "Railway Fuel Cost" before the Western Railway Club recently. The loss in cinders may be so great that a coal of high heating value may in actual firing in the locomotive fire box show a lower yield of heat than a coal of lower heating value. Speaking of tests made at Purdue University, Mr. Endsley said:

> We have found that the heating value of coal as determined in a calorimeter test is not in all cases the same as that obtained in an actual test upon a locomotive; that is, the coal which gave the best results under the calorimeter test did not always give the best results when fired in the locomotive. I have two coals in mind now, Pocahontas coal and Virginia lump coal. Under the calorimeter tests the Pocahontas coal showed a higher heating value, but in the locomotive tests the reverse was true. This result was brought about by the excessive loss from cinders when firing Pocahontas coal, the loss being so great as to decrease its evaporative efficiency and therefore cause it to make a poorer showing. We had but little trouble in firing Pocahontas coal, and in light-power tests we have been able to obtain with that coal an equivalent evaporation of 9 lbs. At heavy-power tests, however, this value was reduced to some extent, the difference being that the loss in heat from cinders was sometimes as high as 20% of the total heating value of the coal. . . . Moreover, the grate losses are not constant for different coals, and this fact has its effect upon results obtained from a practical test of a locomotive.

THE GATUN DAM, on the Panama Canal, is to be built under the followin method of procedure, according to "The Canal Record" of Dec. 4, 1907.

> A channel will be dug through Spillway hill [an elevation between the two branches of the Chagres which the dam will cross.—Ed.] about 300 ft. wide, the bottom of it to be about at sea level. The Chagres river will be passed through this channel during the construction of the dam across the valleys on either side of Spillway hill. As soon as the portions of the dam abutting Spillway hill are high enough to stand 50 to 55 ft. of water in the lake, it is proposed to build across the channel through Spillway hill a concrete dam high enough to hold the lake at the above mentioned level. This will be done for the purpose of facilitating the work of dredging in the Chagres division. During the dry season following the completion of this dredging, the dam across the channel through Spillway hill will be brought up to its full height, and a permanent spillway constructed, which will include the necessary regulating works by means of which the surplus water of the lake will be passed down to the level of the sea.
>
> Preparations will be made for closing the channel through Spillway hill before the Chagres is turned into it and all closures will be accomplished during the dry season, thus enabling the Chagres to form a lake to the height of the dam across the spillway channel during the following wet season. If the present plan of procedure is carried out, and the progress of the work comes up to anticipations, the upper portion of the locks at Gatun will be completed before the dams are high enough to permit the creation of a lake from 50 to 55 ft. in elevation. This lake will make water transportation available for handling large quantities of timber that will be submerged by the lake.

COPPER PRODUCTION IN THE UNITED STATES for 1906 has been summarized, in the "Mineral Resources of the United States, Calendar Year 1906," published by the U. S. Geological Survey, in the following table:

Production of copper:	
Smelter output, lbs.	917,805,682
Value of domestic production of copper	$177,595,888
Mine production, lbs.	916,971,387
Total ore treated, short tons	19,743,000
Average yield of copper, per cent.	2.15
Copper ore treated, short tons	18,000,000
Average yield of copper, per cent.	2.50
Imports, in terms of refined copper, lbs.	215,402,841
Exports, in terms of refined copper, lbs.	446,750,711
Consumption:	
Electrical purposes, lbs.	340,000,000
Brass manufacture, lbs.	210,000,000
Copper sheets, etc., lbs.	35,000,000
Castings, etc., lbs.	100,000,000
Total lbs.	685,000,000
Prices of refined copper:	
Electrolytic, cents per lb.	19.30
Lake, cents per lb.	19.55
Casting, cents per lb.	19.10
World's production, lbs.	1,596,973,700

The total smelter output of 917,805,682 lbs. is a gain of 1.76% over the output of 901,907,843 lbs. in 1905. This is a very small gain compared to the increase of 11% of 1905 over 1904, and of 16.4% of 1904 over 1903. Although mine statistics are not complete before this year it is stated that the gain in mine production was greater during 1906 than that in smelter production. A table showing copper production in the United States since 1883 appeared in Eng. News, Dec. 13, 1906, p. 629.

REINFORCED-CONCRETE VIADUCTS acting as approaches to double-decked reinforced-concrete wharves have been built by the Cala Iron Co. at Seville, Spain. The river Guadalquivir at this point runs through a low-lying country which is subject to high floods, necessitating wharves and docks high above the normal stages of the water and connected to the mainland by high viaducts. These viaducts, designed and constructed by Don J. M. de Zafra, consist of a series of bents 29 ft. on centers and made up of four columns interbraced in pairs, each of which carries a track. These columns are about 9 x 25 ins. in section and are braced together at vertical intervals of about 25 ft. The main girders are 9 x 47 ins. in section and are connected by a concrete slab. The wharves are at two levels, to each of which one of the tracks of the viaduct leads. They are of the regular skeleton column and beam reinforced-concrete construction, covered with a concrete slab. The columns are all founded on reinforced-concrete piles cast on the ground and driven with hammer and water jet when two months old. The viaduct carries loaded ore trains, drawn by 48-ton locomotives, to the wharf where they are unloaded by a special tipping crane. This crane is very heavy and the excessive strains caused by its eccentric action in tipping have necessitated extra heavy diagonal bracing between the columns on the wharves. Two novel features should be noted in this structure. The proportion of cement in the concrete mixture in the piles was varied according to the position of the pile in the wharf. At first wooden forms were used on the work but the excessive heat (often as high as 150° F.) caused them soon to warp out of all safe shape. Thereupon the engineer substituted for all beams and slabs, sheet iron forms, ⅛-in. in thickness and braced with 1¼ x 1¼-in. angles. These had the advantage of a repeated use together with a much smoother finish on the completed concrete. The design of this work is described in some detail in London "Engineering" for Oct. 18, 1907.

EVAPORATION MEASUREMENTS, on a larger scale than ever heretofore attempted, are to be made by the U. S. Weather Bureau in the Salton Basin. A chain of stations will be installed around the Salton Sea, and accurate measurements of evaporation will be made at each station, as well as records of the rate of fall of the sea itself. The area of the sea is now about 440 sq. mi., and the rate of fall of its level, during the years prior to the time when the contents of the basin became highly charged with salt, will furnish a most valuable record of the average evaporation rate in this arid region.

SEVERAL LONG TUNNELS are being built by the Western Pacific Ry. in its low-level crossing of the Sierra Nevada range, the summit of which is nearly 2,000 ft. lower than the summit elevation on the Southern Pacific Ry. This lower summit is reached in the Beckwourth Pass, and is below the snow line. There are about 43 tunnels, aggregating 8½ miles; the longest are the following: Spring Garden tunnel, near Oroville, Cal., 7,200 ft.; Beckwourth Pass tunnel, 6,000 ft.; Flower Lake tunnel (in Nevada), 5,664 ft., and Niles Canyon tunnel, 4,000 ft.

THE ELIZABETH LAKE TUNNEL, in the line of the Los Angeles Aqueduct, is now being driven from both ends, work having been started in September. The tunnel will be 27,216 ft. long, and is estimated to cost $1,913,200, or about $75.50 per lin. ft. A total drop of 1,500 ft. in the aqueduct line near the tunnel will be used for power development. Mr. Wm. Mulholland is Chief Engineer and Mr. J. B. Lippincott, M. Am. Soc. C. E., is Assistant Chief Engineer of the Los Angeles Aqueduct, Los Angeles, Cal. The aqueduct, as projected, was described at length in Engineering News of Jan. 24, 1907.

PERSONAL.

J. W. Ager, formerly Electrical Aide in the U. S. Bureau of Yards and Docks, has become Manager of the southern office of Muralt & Co., Consulting Engineers with headquarters at Birmingham, Ala.

Mr. C. F. Breitze, formerly Assistant Engineer with Messrs. Hering and Fuller, Consulting Engineers, has accepted a position in the Engineering Department of the New York State Board of Health.

Mr. Arthur Williams, Assoc. M. Am. Inst. E. E., General Inspector, The New York Edison Co., has recently been honored by the French Republic with the decoration Officier de l'Instruction Publique.

Mr. Austin L. Bowman, M. Am. Soc. C. E., has recently been appointed a Consulting Engineer in the Department of Bridges of New York City to fill the vacancy left by Mr. Henry B. Seaman, M. Am. Soc. C. E., who became Chief Engineer of the Public Service Commission of New York City.

Mr. Seaton M. Scott, M. Am. Soc. M. E., who has been engineer in charge of the construction of the Waterside 80,000-KW. Station of the New York Edison Co., will sever his connection with that company on Dec. 31, as the work with which he was connected is now practically completed.

Mr. A. D. Chidsey, Jr., has been appointed Resident Engineer of the Engineers Committee on the Elimination of Grade Crossings in Wilkesbarre, Pa., to take effect Jan. 1, 1908. The above committee consists of the City Engineer of Wilkesbarre and the Chief Engineers of the Pennsylvania R. R., Central Railroad of New Jersey, Lehigh Valley R. R., and Delaware & Hudson R. R., respectively.

Mr. Mansfield Merriman, M. Am. Soc. C. E., for many years Professor of Civil Engineering at Lehigh University and now in consulting practice in New York City, has sailed for Europe to be absent several months. Letters may be addressed to him in care of Morgan, Harjes et Cie., 31 Boulevard Haussmann, Paris, France, and printed matter may be sent to him in care of Thaddeus Merriman, Essex Falls, N. J.

Col. Geo. W. Goethals, Chairman and Chief Engineer of the Panama Canal Commission, arrived in Washington on Dec. 18, and will spend the next two months after conferences with Congressional committees on the canal are completed, on a vacation rest. Regarding the proposed increase of width of the canal locks beyond the 100 ft. now proposed, Col. Goethals is quoted as follows by the daily press:

> Colonel Goethals says that so far as is known at present a width of 100 ft. for the locks will accommodate mercantile shipping for years to come, and that 100 ft. is ample for all naval construction now existing, building or proposed. A naval vessel 98 ft. beam would be 8 ft. wider than the latest battleship proposed, and could get through a 100-ft. lock without trouble.
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> The question of increasing the width of the locks is one that more properly belongs to the navy to determine, it is suggested by him, and if the efficiency of a battleship that can use the 100-ft. lock is materially less than one requiring 110 ft. width, there is no question that the 110-ft. lock should be adopted. It is pointed out that the questions involved in any widening of the locks are increased cost and the longer time required for filling, thereby decreasing the capacity of the canal and the effect on the water supply for it.
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> It is declared that the commission is ready to do whatever the President and his naval advisers think best, as the work at present is in such shape that the increase in width can be made without any detriment to the work or undoing any which has thus far been done.

Obituary.

Wm. F. Love, M. W. S. E., for several years Chicago representative of the Wheeler Engineering Co., died at New York Dec. 13, aged 42.

Elisha M. Hagler, a structural engineer of St. Louis, Mo., died at his home on Dec. 8, aged 32 years. He was a graduate of the engineering department of the University of Iowa and had served with the Iowa volunteers in the Philippines. He is survived by a widow.

William Thomson, Lord Kelvin, died on Dec. 17, 1907, at Glasgow, Scotland, at the age of 83, as noted in our last issue. He was, at least for the last two decades of his life, the most prominent figure in an illustrious group of English mathematicians and physicists, of whom Lord Rayleigh is the chief remaining member. Born on June 26, 1824, in Belfast, Ireland, as the son of James Thomson, professor of mathematics, who later went to the College of Glasgow, William Thomson was endowed with hereditary ability in high degree. A thorough education, together with remarkable precocity, brought him to Glasgow as professor of natural philosophy at the University of Glasgow in 1846, when he was only 22. Here he remained for 53 years, retiring only in 1899. Even before appointment to this position, however, he had written several noteworthy papers on physical problems, mainly in the theory of heat. The major portion of his life work was connected with the investigation of theoretical physics, principally in the theory of heat and of electricity and magnetism, in both of which fields his labors were profoundly influential in establishing truer theoretical conceptions and extending the structure of mathematical theory. Among present day conceptions which may be said to be due to Lord Kelvin is the famous theory of the indestructible vortex-ring atom. In the study of the atom he also entered deeply into the theory of heat and did much toward establishing a correct knowledge of the relation between heat and work.

Kelvin was closely connected with the two first Atlantic cables, that of 1858 and that of 1866, and later was electrical engineer for the French Atlantic cable (1869), the Brazilian and River Plate (1873), the West Indian cables (1875) and the Mackay-Bennett Atlantic cables (1879). The mirror galvanometer and the siphon recorder are among the devices invented by him for these cables; his contributions were indeed vital to the ultimate success of transatlantic telegraphy. Numerous other instruments, mainly electrical ones, were devised by him, and experimental work in electricity still bears the impress of the early work of William Thomson. His writings consist mainly of his many scientific papers, contributed to various societies. These have been collected in several series. Only one book was written by him, and this was produced in collaboration with Professor Tait, Thomson & Tait's "A Treatise on Natural Philosophy," first published in 1867. On account of his distinguished services in conection with the transatlantic cables in 1866 he was made Sir William Thomson, and for his well directed efforts in the whole scientific field he was created Baron Kelvin in 1892. His title died with him. He was honored by many scientific bodies and was decorated by half a dozen European governments. His body is to be buried in Westminster Abbey.

ENGINEERING SOCIETIES.

COMING MEETINGS.

AMERICAN ECONOMIC ASSOCIATION.
Dec. 28-31. Twentieth annual meeting at Madison, Wis. Secy., Winthrop M. Daniels, Princeton, N. J.

AMERICAN ASSOCIATION FOR THE ADVANCEMENT OF SCIENCE.
Dec. 30-Jan. 4. Annual meeting at University of Chicago. Secy., L. O. Howard, Cosmos Club, Washington, D. C.

MICHIGAN ENGINEERING SOCIETY.
Jan. 7-8. Annual meeting at Battle Creek, Mich. Secy., Alba L. Holmes, 574 Wealthy Ave., Grand Rapids.

IOWA ENGINEERING SOCIETY.
Jan. 8-9. Annual meeting at Des Moines. Secy., A. H. Ford, Iowa City.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
Jan. 15. Annual meeting at New York City. Secy., Chas. Warren Hunt, 220 West 57th St., New York City.

ILLINOIS SOCIETY OF ENGINEERS AND SURVEYORS.
Jan. 15-17. Annual meeting at Champaign, Ill. (University of Illinois). Secy., E. E. R. Tratman, 1636 Monadnock Block, Chicago.

INDIANA ENGINEERING SOCIETY.
Jan. 16-18. Annual meeting at Indianapolis. Secy., Charles C. Brown, 408 Commercial Club Building, Indianapolis.

NATIONAL ASSOCIATION OF CEMENT USERS.
Jan. 20-25. Fourth annual convention at Buffalo. President, Richard L. Humphrey, 1001 Harrison Bldg., Philadelphia, Pa. Convention Manager, Dal H. Lewis, 760 Main St., Buffalo.

WOOD PRESERVERS' ASSOCIATION.
Jan. 21. Fourth annual meeting at Kansas City. Secy., C. W. Berry, Laramie, Wyo.

AMERICAN SOCIETY OF HEATING AND VENTILATING ENGINEERS.
Jan. 21-23. Annual meeting at New York, Engineering Societies Building, 29 West 39th St. Secy., W. M. Mackay, 29 West 39th St., New York.

NATIONAL SOCIETY FOR THE PROMOTION OF INDUSTRIAL EDUCATION.
Jan. 23-25. Annual meeting at Chicago. Secy., C. R. Richards, Columbia University, N. Y.

CANADIAN SOCIETY OF CIVIL ENGINEERS.
Jan. 28-30. Annual meeting at Montreal. Secy., Prof. C. H. McLeod, 877 Dorchester St., Montreal.

OHIO ENGINEERING SOCIETY.
Feb. 11-13. Annual meeting at Columbus. Secy., E. G. Bradbury, 85 North High St., Columbus.

THE CHICAGO AND SUBURBAN HEALTH LEAGUE was organized at Chicago on Dec. 7. Meetings will be held quarterly. Dr. Wm. R. Parkes, of Evanston, was elected President, and Dr. Heman Spalding, of Chicago, was elected Secretary and Treasurer.

THE AMERICAN SOCIETY OF MECHANICAL ENGINEERS will hold its next monthly meeting at the Engineering Societies Building, New York, Jan. 4. The paper of the evening will be read by Mr. H. M. Dixon, President of the Safety Car Heating and Lighting Co., upon "Car Lighting." There will be in operation exhibits of different methods such as the Pintsch method, the vapor mantle system, a new acetylene system and several varieties of axle lighting with their regulating and governing mechanism.

AMERICAN SOCIETY OF CIVIL ENGINEERS. A lengthy session on the evening of Dec. 18 was devoted to a paper and discussion on "Municipal Refuse Destruction: An Investigation." The paper was by Mr. J. T. Fetherston, Assoc. M. Am. Soc. C. E., Superintendent of Street Cleaning, Borough of Richmond, New York City. (See Proc. Am. Soc. C. E., November, 1907, for paper, with numerous tables and illustrations). The paper consisted of (1) a summary of refuse-burning data up to 1904; (2) a review of physical, calorific and chemical analyses of refuse collected on Staten Island under the direction of the author, and of experiments with an old garbage furnace; and (3) detailed accounts, mostly tabular, of 40 refuse destructors in Great Britain, visited by the author in 1906. As a result of Mr. Fetherston's investigations and of his recommendations based thereon a refuse destructor of the British type is now nearing completion at West New Brighton, Borough of Richmond.

Mr. Foster Crowell, M. Am. Soc. C. E., who has recently been appointed Commissioner of Street Cleaning for New York City, on being called on to open the discussion, characterized as primitive many of the methods of his department, as found by him on taking office. Riker's Island, in the East River, is being enlarged by means of ashes, etc., from an area of 87½ to 151 acres, and it is proposed to extend the bulkhead lines to enclose 360 acres. The fill with city wastes is carried to 65 ft. above water, with a probable shrinkage of 50%. Mr. Crowell suggested, as a possible desirable substitute for this method of disposal, that the mixed refuse be burned, the heat of combustion utilized and the residue, which would then be wholly inorganic, used as filling. Mr. Fred. L. Stearns, Assoc. M. Am. Soc. C. E., of the New York Street Cleaning Department, expressed himself strongly in favor of reduction for garbage and cremation for mixed wastes. He described briefly the many different small furnaces tried by the department and made a number of sarcastic comments on the unnecessarily high labor cost of burning garbage in New York, due to "soldiering," etc.

Mr. Wm. Mayor Venable, Assoc. M. Am. Soc. C. E., sent a written discussion. He advanced arguments in favor of burning garbage separately, and in support of the American garbage furnaces. Mr. B. F. Welton, Assoc. M. Am. Soc. C. E., described calorific tests of refuse made by him at the Lederle Laboratories for Mr. Fetherston, and also chemical tests, made by Prof. Stephen F. Peckham. The two lines of tests checked very well. Mr. Albert A. Carey, M. Am. Soc. M. E., discussed the technical aspects of calorific tests, and the difficulties of burning wet material. He had satisfactorily reduced the moisture in wet material by passing it through rough-faced rolls, one larger than the other, revolving at the same speed, and arranged to spread so as to allow the passage of lumps. The range of furnace temperatures suggested by Mr. Fetherston, 1,250 to 2,000° F., Mr. Carey thought too low. A maximum of 2,500 would be better, and could be maintained without undue expense for furnace repairs if the best refractory linings and the best furnace masons were employed in construction.

Mr. Leash, an engineer representing Heenan & Froude, constructors of the New Brighton furnaces, doubted whether American engineers had anything to do with the design of American garbage furnaces. He thought American and foreign refuse differed little in calorific value. Mr. L. L. Tribus, M. Am. Soc. C. E., Consulting Engineer to the Borough President of Richmond, said that local conditions on Staten Island, present and prospective, made it particularly desirable to have there a central all-consuming, self-destroying means of refuse disposal.

At the conclusion of the session (about 11 p. m.), it was announced that the next summer convention, which will be held at Denver, will probably come a week earlier than was intended, making it fall the last week in June. The next regular meeting of the society, in New York, will be held on Jan. 8. In place of a formal paper there will be an informal discussion on "Reinforced Concrete in Engineering Construction," opened by Mr. E. P. Goodrich, M. Am. Soc. C. E., of New York City.